U.S. Department of Transportation

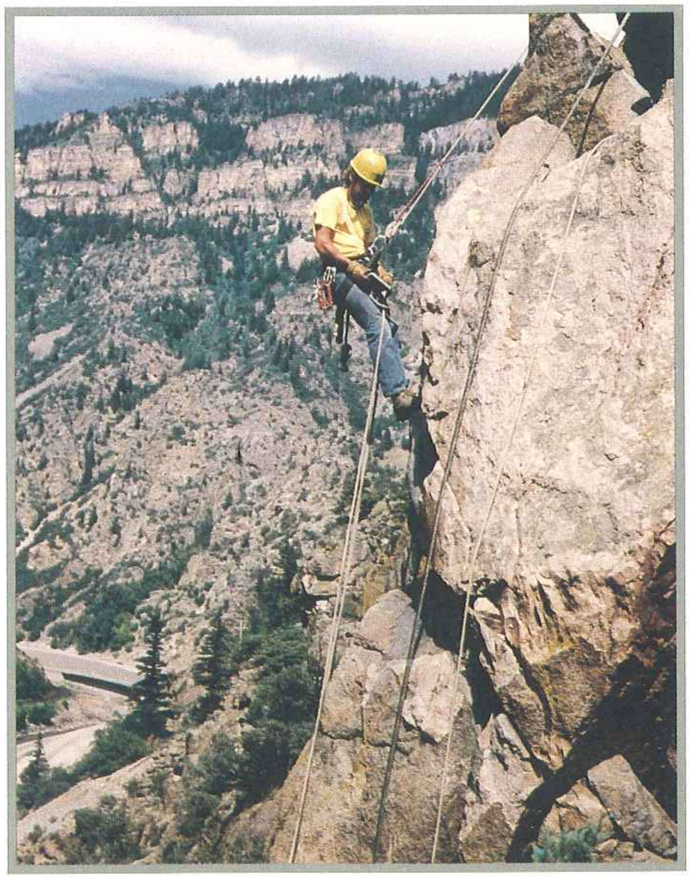
Federal Highway Administration

Publication No. FHWA SA-93-085
March 1994

NHI Course No. 13219

Rockfall Hazard Mitigation Methods

Participant Workbook



NHI National Highway Institute

TECHNOLOGY APPLICATIONS

Innovation Through Partnerships

PARTICIPANT WORKBOOK
FOR
ROCKFALL HAZARD MITIGATION METHODS

C.O. Branwer, P.Eng.
C.O. Brawner Engineering Ltd.

Prepared for
Federal Highway Administration

Contract Number
DTFH61-92-Z-00069

SNI International Resources, Inc.

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EXECUTIVE SUMMARY

The FWHA is developing a series of highway manuals to provide up to date technology and to provide uniformity of design and maintenance across the United States. This FWHA *Manual of Practice on Rockfall Hazard Mitigation Methods* has been developed as a reference for State highway engineers for design, construction, and correction of rockfall problems. It has been developed in response to an increase in litigation cases due to rock instability where vehicles have been damaged or there has been injury or loss of life to vehicle occupants.

Rockfall is caused by many factors, most notably adverse structural geology, groundwater-related problems or improperly designed and controlled blasting procedures during construction. The various causes are described.

Types of rockfall are influenced by the structural geology and orientation of the discontinuities relative to the highway cut slope. Determination of the type of rockfall significantly influences the method of mitigation or stabilization. Procedures that will stabilize one type of failure may not be successful for another type. This manual describes and illustrates the important types of failure.

In order to assess existing or potential rockfall problems, various investigations are described. They include the use of existing maps, air photos and reports, climatic data, field reconnaissance, and field monitoring. Maintenance history, traffic volumes, and alignment considerations that impact accident potential and risk must be assessed. Field geotechnical investigations include determination of rock type, structural geology, discontinuity roughness and evidence of past instability. Water chemistry tests will determine if corrosion of metal components will be a problem. Field tests where rocks roll rock down or below rock slopes assist in rock protection design. A review of investigation programs is presented.

In some instances, the potential for rockfall is known from experience, or the potential can be evaluated from site conditions. Geotechnical monitoring employing a variety of observational or measurement techniques is a major tool used to evaluate stability.

The development of automatic remote electronic distance measurement (EDM) and Global Position System (GPS) satellite surveying has greatly improved field monitoring capability to predict impending failure.

Where rock cliffs exist above highway slopes or where rock may bounce down a slope, the analysis of rockfall energies and trajectories is necessary to assist in the design and location of protective measures. Washington, Oregon, Colorado, and California have been leaders in developing this technology. Original rock rolling experiments and the resultant design charts developed by the State of Washington have been enhanced by recent computer simulation. Presently, the Colorado Rockfall Simulation Program (CRSP) is the best computer simulation program available and is described in some detail. It must be treated as a tool to assist the design.

A great variety of rockfall mitigation procedures are available. They are very site specific and the procedure selected is dependent on many factors. Procedures include (a) Stabilization, (b) Protection, and (c) Warning Systems. The important factors that influence the remedial procedure include rockfall history, existence of adverse structural geology, existing ditch design, traffic risk, maintenance costs, and remedial costs. The more common stabilization methods include excavation, scaling, slope drainage, shotcrete, buttresses, dowels, rock bolts, and cable lashing. Protection methods include relocation, ditch improvement, mesh on the slope, catch fences, catch walls, rock sheds, and tunnels. Warning methods include monitoring rock movement, and installing electric wire fences and warning signs. Traffic patrols during and following heavy precipitation are recommended. Details of each method are outlined.

A specific problem is the use of procedures to protect traffic during the stabilization or construction of mitigation or measures. A variety of programs, including traffic control, detours, and special catchment structures, are described.

Enviornmentalists have expressed concern about the appearance of rock stabilization structures or systems, as well as the appearance of rock faces where controlled (preshear or cushion) blasting has been used. Government policy dictates that the prime consideration for highway operation is

SAFETY. It *shall not* be compromised. The 1991 Intermodal Surface Transportation Efficiency Act (ISTEA) mandates the following: "It shall be the national policy to bring all of the Federal Aid Systems up to standard and increase the safety of those systems to the maximum extent." Cost-effective aesthetic treatments are available that can be incorporated on some rock projects, if deemed appropriate. For example, colored concrete structures, colored plastic-coated fencing, special structural surface finishes, rock staining, and planting small shrubbery may have application. Where safety will not be reduced, and where excessive costs will not be involved, the input of landscape architects, responsive government agencies, and rational members of the public should be encouraged to have input into final design.

The development of specifications and construction requirements for rockfall mitigation or stabilization is very specialized and site specific. Typical example specifications for the most common mitigation and stabilization procedures have been included. They will generally require some changes to suit any specific site. Where possible, unit rates bid prices should be used for payment. However, due to the difficult nature of many mitigation and stabilization projects, the contracts must allow flexibility with some components of the work to be done on a time and materials (cost plus) basis.

It must also be emphasized that each State will have precedent and preference for the development of specifications, format, content, and procedures.

To illustrate the range of actual rockfall problems, 12 case examples have been described by members of the Technical Review Panel and the author. It is the intent that these examples will be published separately and augmented as more examples come available. Some examples describe rockfall occurrences with attendant damage, injury, and loss of life. Other examples describe the design to mitigate or stabilize potential rockfall conditions.

The procedures, difficulties, and traffic control restraints impose severe controls on costs of rockfall mitigation and stabilization. Since a great many factors influence the cost, there is a wide range in costs for various procedures. Several States were requested to provide cost information from past projects. To cover the spectrum, low-range and high-range figures were requested. The cost-comparison table was developed so comparisons among states could be made.

The legal and liability issues of accidents related to rockfalls are addressed in chapter 12. Since the law varies in each State, legal departments from the States of California, New York, North Carolina, and Washington provided a summary of State legal precedent, policy, or case examples. The statements presented are State-specific. It is apparent that a variety of considerations will impact litigation decisions. They include:

- Standard of design and construction when the highway was built.
- Ability to detect and evaluate rockfall potential.
- Adequacy of highway maintenance in areas of rock slopes.
- Type and location of warning signs.
- The State's financial capability to maintain rock slope stability throughout the state.
- State personnel using rock stability evaluation techniques that meet the Standard of the Industry.

The United States Code Title 23 Highways was updated as a 1992 edition. All States now are obligated and must be familiar with the safety aspects of this Code.

ACKNOWLEDGEMENTS

The author acknowledges the contribution of a Technical Review Panel, which included the following members:

- Ron Chassie, Senior Geotechnical Engineer, FWHA, Portland.
- Barry Siel, Geotechnical Engineer, FWHA, Denver.
- John Duffy, Geologist, California Department of Transportation.
- Rich Andrew, Geologist, Colorado Department of Transportation.
- Alexander Yatsevitch, Engineering Geologist, New York Department of Transportation.
- Richard Cross, Geologist, New York State Thruway Authority.
- John Ledbetter, Engineering, North Carolina Department of Transportation.
- Steve Lowell, Chief Engineering Geologist, Washington Department of Transportation.
- Larry Pierson, Chief Engineering Geologist, Oregon Department of Transportation.

Special acknowledgement is also given to the following individuals:

- John Duffy, State of California, who prepared the Chapter on Rockfall Analysis.
- Stewart Anderson, graduate student at the University of British Columbia, who assisted with the literature search, draft outline, references, and glossary.
- Lupe Carbajal, SNI International International Resources, Inc., Phoenix, Arizona, who supervised layout and publication of the contracted draft and final manual.

Acknowledgement of the States that contributed to a survey of rockfall problems and mitigation procedures is also made. The summary of this survey is provided in Table 1.

CONVERSION FACTORS

	Imperial	Metric	SI
Length	1 mile	1.609 km	1.609 km
	1 ft	0.3048 m	0.3048 m
	1 in	2.54 cm	25.40 mm
Area	1 mile^2	2.590 km^2	2.590 km^2
	1 acre	0.4047 hectare	4046.9 m^2
	1 ft^2	0.0929 m^2	0.0929 m^2
	1 in^2	6.452 cm^2	6.452 cm^2
Volume	1 yd^3	0.7646 m^3	0.7646 m^3
	1 ft^3	0.0283 m^3	0.0283 m^3
	1 ft^3	28.32 litres	0.0283 m^3
	1 Imperial gallon	4.546 litres	4546 cm^3
	1 US gallon	3.785 litres	3785 cm^3
	1 in^3	16.387 cm^3	16.387 cm^3
Mass	1 ton	1.016 tonne	1.016 Mg
	1 lb	0.4536 kg	0.4536 kg
	1 oz	28.352 gm	28.352 gm
Density	1 lb/ft^3	16.019 kg/m^3	16.019 kg/m^3
Unit weight	1 lbf/ft^3	16.019 kgf/m^3	0.1571 kN/m^3
Force	1 ton f	1.016 tonne f	9.964 kN
	1 lb f	0.4536 kg f	4.448 N
Pressure or stress	1 ton f/in^2	157.47 kg f/cm^2	15.44 MPa
	1 ton f/ft^2	10.936 tonne f/m^2	107.3 kPa
	1 lb f/in^2	0.0703 kg f/cm^2	6.895 kPa
	1 lb f/ft^2	4.882 kg f/m^2	0.04788 kPa
	1 standard atmosphere	1.033 kg f/m^2	101.325 kPa
	14.495 lb f/in^2	1.019 kg f/cm^2	1 bar
	1 ft water	0.0305 kg f/cm^2	2.989 kPa
	1 in mercury	0.0345 kg f/cm^2	3.386 kPa
Permeability	1 ft/year	0.9659 x 10^{-6} cm/s	0.9659 x 10^{-8} m/s
Flow rate	1 ft^3/s	0.02832 m^3/s	0.02832 m^3/s
Moment	1 lbf ft	0.1383 kgf m	1.3558 Nm
Energy	1 ft lbf	1.3558 J	1.3558 J
Frequency	1 c/s	1 c/s	1 Hz

SI Unit Prefixes

Prefix	tera	giga	mega	kilo	milli	micro	nano	pico
Symbol	T	G	M	k	m	μ	n	p
Multiplier	10^{12}	10^9	10^6	10^3	10^{-3}	10^{-6}	10^{-9}	10^{12}

SI Symbols and Definitions

N	=	Newton = kg m/s^2
Pa	=	Pasacal = N/m^2
J	=	Joule = m.N

CHAPTER 1

INTRODUCTION

Rockfalls along highways occur where natural slopes or rock excavations exist. When such falls reach the roadway they are a hazard to travelling vehicles. Tens of millions of dollars are spent annually on rockfall and rock slope maintenance. Annual legal claims resulting from rockfalls also are in the millions of dollars annually. Several States have had traffic deaths caused by rockfall.

Most highway agencies with rock cuts are confronted with the rockfall problem, including the development of stabilization measures.

This manual has been contracted by the FHWA to provide a current review of rockfall stabilization (mitigation and remediation), causes and types of failures, and investigations with emphasis on construction procedures, specifications, contracting procedures, and inspection, payment, and acceptance. Other issues were to include environmental policy, evaluation of site conditions, stabilization costs and legal and liability questions. Design aspects were addressed in Rock Slopes, November 1981.

This manual is intended as a companion to the Rockfall Hazard Rating Manual, which was developed at the same time. Other manuals in the FHWA series include Rock Slopes (1989), Rock Blasting and Overbreak Control (1991), and Highway Slope Maintenance and Slide Restoration (1988).

This manual is intended to be a reference manual for highway agency engineers for design, construction, and correction of rockfall problems.

1.1. DEFINITION

For the manual, the term rockfall is defined as, "The movement of rock of any size from a cliff or other slope that is so steep the mass continues to move down slope. Movement may be by free falling, bouncing, rolling or sliding. The fall may involve more than one rock but does not include large volumes of rock, rock avalanches, or landslides including rock (Caltrans, 1985). Rockfall can be a continuous process over a considerable time period or a single or series of single, intermittent events. Rockfall can be initiated by many means (Andrew, 1992).

1.2. SIGNIFICANCE OF ROCKFALLS

Accidents caused by a rockfall are shown in Figs. 1-1 to 1-4. Accidents can occur when a vehicle cannot stop in time and strikes rock on the highway or the vehicle may be struck directly by falling rock.

Rockfall is caused by many factors, including unfavourable structural geology, adverse groundwater-related conditions, poor blasting practices during original construction or reconstruction, climatic changes, weathering, and tree levering. Many types of failure also can occur, such as planar, wedge, circular, block, toppling, buckling, key block, and ravelling.

Recent investigation, design and construction procedures have led to improved stability of rock slopes. However, practically all states have roads with existing rock slopes that were developed decades ago. These slopes have deteriorated with time and now create serious, ongoing maintenance and stabilization problems. In recent years, improved grade and alignment requirements have resulted in very high cuts to more than 300 feet (91.5 meters) in some cases. At other locations, very high natural rock slopes or cliffs exist adjacent to the highway. Rockfall from such high slopes is dangerous because of greater energy and run-out potential.

Stabilization techniques have evolved over time. They include scaling, which is required for most slopes before any other programs are implemented. Other procedures involve the installation of dowels, bolts, shotcrete, mesh, wire catch fences, buttresses, catch walls, and the development of adequate catch ditches.

Where many rock cuts exist in the State, a rating system is required to establish a hazard rating and priority stabilization program. The companion manual, Rockfall Hazard Rating System, deals with this problem. In order to evaluate priority, site investigations, including assessment of geologic conditions, traffic volume, sight distance, rockfall occurrence, and maintenance history, must be performed.

Figure 1-1. A Classic Chevrolet auto crushed by a rockfall on the outer highway shoulder. The fall occurred during heavy precipitation in 1989 *(Courtesy Oregon Department of Transportation)*.

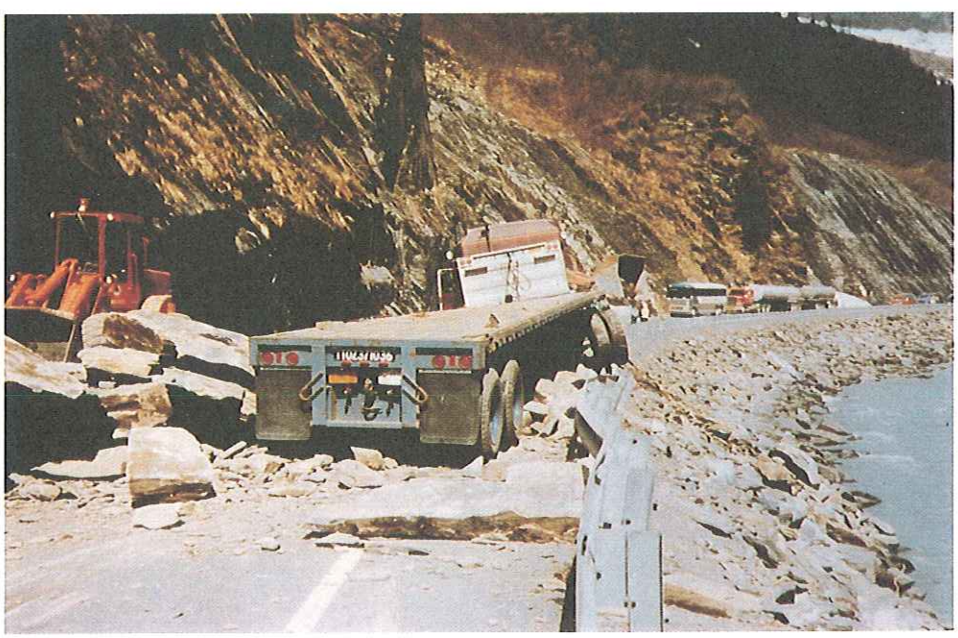


Figure 1-2. An empty truck collided with a 20 to 25 yd^3 (15.3 to 19m^3) rockfall in Keystone Canyon, Alaska in 1991 *(Courtesy Alaska State Highway Patrol)*.

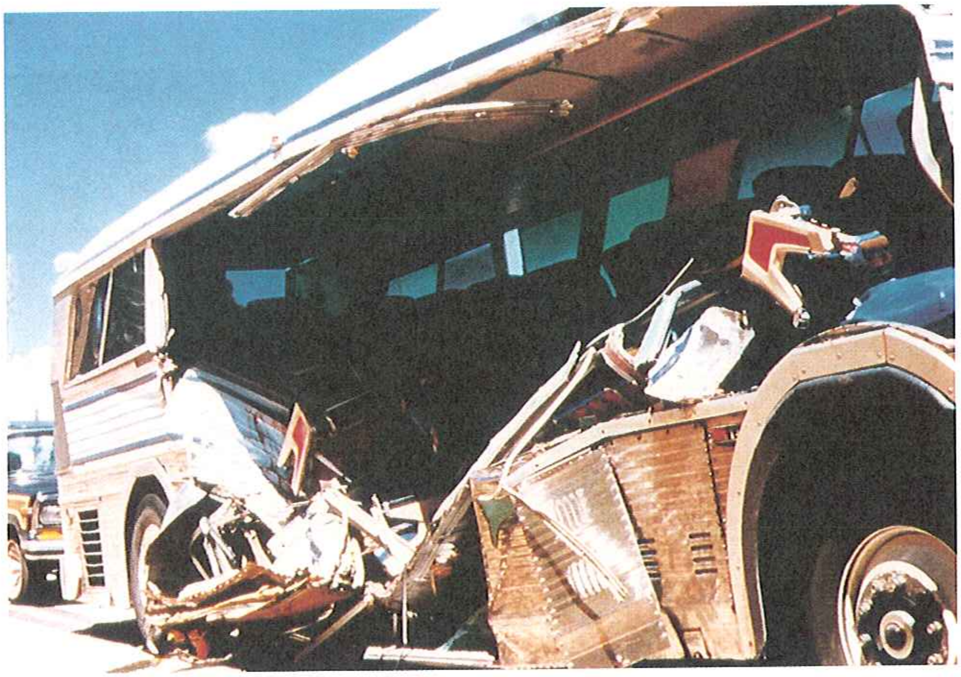


Figure 1-3. Eight passengers were killed and others were injured when a bus was struck by large rockfall *(Courtesy Colorado Department of Transportation).*

Figure 1-4. Car hit by rockfall on North Carolina Highway I-40 *(Courtesy N.C. Department of Transportation).*

Theoretical and modelling studies may be helpful for ditch and catch fence design. The original concept is based on practical research performed by Ritchie (1963). Today, advantage can be taken of energy analysis, rockfall trajectory analysis, and powerful computer modelling techniques such as the Colorado Rockfall Simulation Program. These procedures, however, must recognize that the occurrence of a "bad or freak bounce" is not encompassed by the models. Whenever possible field correlation with rolling rock tests is recommended.

The ongoing implications of the economics of rockfall is important. Rock slopes deteriorate with time due to weathering, freeze-thaw, wet-dry and hot-cold cycles, and erosion conditions. As traffic volumes increase, the potential for vehicle damage and public hazard increases. Indirect costs can result from traffic delays and detours, extra engineering costs, legal fees, and medical costs.

The legal implications also are becoming more important. In the last ten years the author has been involved in eight lawsuits or inquiries in the Pacific Northwest involving over $10 million, eight deaths and three injuries. Effective stabilization programs must be implemented to minimize rockfall accidents. A wide range of procedures to mitigate potential instability and to remediate rockfall problems is essential since each problem is site specific.

1.3. ROCKFALL MITIGATION POLICY

Each State has site specific rockfall conditions that are influenced by many conditions. These include geology, climate, topography, traffic volume, economics, and environment. It is very important that a rockfall mitigation policy be developed to establish procedures, priorities and economics for each State. A proactive rather than a reactive approach is preferable.

A major problem is that rockfalls are difficult to monitor-that is difficult to predict when the rockfall potential has become serious. In addition, isolated rockfalls occur rapidly. Ongoing record keeping of all rockfalls in rock cuts will assist in defining rockfall-prone areas. Annual inspections, stability, and hazard rating are essential. In the long term, a systematic inspection, priority rating, and stabilization program will reduce accidents, costs and liability.

A rational and systematic rockfall hazard rating and stabilization program will require the implementation of an agency policy to define priorities. The legal implications of death, vehicle damage, and traffic disruption dictate that safety must be the major priority. Economic limitations will be an important priority for most sites. Where feasible, environmental and aesthetic concerns can be addressed provided they do not compromise safety, and can be economically justified.

It must be recognized that with the thousands of rock cuts in the U.S., many of which are decades old, that 100 percent control of rockfalls is impossible. A national systematic program to rate rock cuts according to a hazard priority is developing and expanding. A major requirement is to maintain comprehensive rockfall records.

Specific rockfall control procedures must also be developed to protect traffic during reconstruction of existing highways. Such procedures may be unique and expensive as compared to nonconstruction zones. They must be evaluated and compared to the development and cost of detours.

All rockfall incidents should be investigated and recorded. Lessons learned from each incident should be documented. There usually is more to learn from failures than from successes.

The costs of mitigation and remediation vary widely. Attempts are made in the manual to quantify the general cost range for various procedures and conditions. A major impact will be the ease of access to the site. For example, the use of helicopters to airlift men, supplies and equipment may be required in extreme conditions.

This manual is intended to provide practical assistance and procedures to deal with most rockfall problems. There will be ongoing improvement in investigation, evaluation and stabilization technology. It is recommended this manual be reviewed and updated within a reasonable time period.

The development of the present manual has been enhanced by information and by case examples provided by many transportation departments across the United States. Varying conditions of geology, rock quality, climate, traffic, and site access are encompassed.

An international literature search and review has been performed to provide additional technical information, experience, and stabilization concepts.

A questionnaire was developed to request information from all States on rock stability problems and mitigation procedures. Twenty-three States indicated rock slope stability is an ongoing problem and provided information. Table 1-1 provides a summary of those responses.

Table 1-1.

Summary of State Responses to Rock Stability Questionnaire, Number of Responses: 23.

Q2: What are the most frequent causes of failure?

Adverse Structural Geology	16	Construction Blasting	6	Weathering	21
Weak Rock	16	Vibration – Seismic	1	Tree roots	3
Groundwater	10	Stress Relief	5	Erosion	13
Other	2				

Q3: What types of failure have you had?

Circular	9	Block	17	Key Block	2
Planar	15	Toppling	12	Boulder Fall	19
Wedge	16	Ravelling	17	Other	4

Q4: What investigations have been performed to evaluate rock falls?

Visually Examine Slope	22	Review History	13	Field Trials – Roll Rock	9
Geologic Mapping	13	Seepage – Ice	2	Mountaineer Inspection	5
Traffic Implications	4	Water Chemistry – Corrosion	2	Other	5
Road and Slope Geometry	13	Photos – Stereo	8		

Q5: What Monitoring techniques have been used?

Record Rockfall History	13	Hubs and Measuring Bar	5	Warning Fences	1
Survey Stakes in Cracks	9	Tilt Plates	1	Warning Systems–Sirens, Lights	2
Tripod Wire Monitors		Extensometers	3	Other	6
EDM – Mirror Measurements	4	None	6		

Q6: Have you used special techniques to monitor rockfalls?

Energy Analysis	3	Field Testing	10
Rockfall Trajectory Analysis	8	Other	2
Rockfall Computer Simulation	6		

Q7: What rockfall mitigation methods have you used?

Excavation	16	Rock Shooting	12	Draped Mesh	11
Rockfall Barriers	13	Walls	12	Buttresses	7
Dissipation Mounds	4	Drainage	6	Construction Controlled Blasts	14
Rock Anchors	9	Shotcrete	10	Relocation	7
Scaling	16	Dowels	7	Ditch Design	18
Rock Fences	13	Tunnels		Warning(signs–radio)	7
Rock Sheds		Other	4		

Q8: Contract Provisions

Rock Excavation	9	Shotcrete	4	Wire Mesh	7
Rock Blasting	7	Dowels	2	Protection Fences	2
Scaling	6	Rock Bolts	3	Rockfall Barriers	1
Slope Drainage	2				

CHAPTER 2

CAUSES OF ROCKFALL

2.1. INTRODUCTION

In order to develop remedial measures to prevent or substantially reduce the risk of rockfall hazard, it is important to recognize the active elements that can generate rockfalls. There are numerous conditions that can affect a slope and produce a rockfall hazard at a site. In most instances, more than one factor contributes to the failure. These factors can be grouped into two dominant categories: the rock mass characteristics and the type of rock mass failure. The latter is discussed in detail in chapter 3.

The characteristics of the rock mass are an important criteria for the recognition, evaluation, and control of a potential hazard. Some of the characteristics to be discussed in this chapter are the influence of structural discontinuities, rock type, weathering and alteration, groundwater, and external stress. Other characteristics may occur that are not a direct result of natural processes but are a product of human activity. Examples of this are the effects of poor blasting practices to develop rock slopes, common prior to about 1980, or vibration from trains, heavy equipment, or stress relief due to excavation.

2.2. STRUCTURAL DISCONTINUITIES

If all rock was intact and massive, even with low strength, rock slopes would stand vertical for thousands of feet. In practice, this does not occur in the majority of cases because discontinuities in the rock mass result in planes or zones of weakness. Failure is likely if these planes or zones are oriented, singly or in combination, with a dip angle out of the slope at an angle steeper than the effective angle of friction within the discontinuity (friction plus the influence of surface roughness) or influenced by pressure due to water, ice, wind, or vibration.

The stability of any rock or rock slope is usually dependant upon the characteristics of the discontinuities. As a rule, the physical and mechanical properties of the rock mass are largely dependant upon the orientation, location, and disbursement of the planar features. Therefore, the stability of a rock or rock slope is primarily assessed through analysis of the discontinuity characteristics and their relationship with the slope orientation and inclination.

Discontinuities comprise a number of features. The more common are:

Foliation: Parallel layers of flow cleavage or schistosity, due to parallelism of constituent minerals, for example, mica.

Bedding: Parallel layers of rock of different textures and often different colors, with planes between layers.

Joints: Well-defined cracks in a rock along which little or no slippage has occurred. Joints frequently divide rock into blocks.

Faults: A fracture along which there has been slippage of the contiguous masses against one another. The slippage may result from compression, tension, or torsion. As the length of the fault increases, the width also generally increases. Fault gouge is generally finer in clayey-type rock and coarser in granular rock. The shear strength is less for clayey-type gouge. Narrow faults are described as shears. Clayey-filled faults generally act as aquitards to restrict seepage, whereas faults filled with granular material may enhance seepage.

Photographs of typical discontinuities are shown in Figures 2-1 to 2-5.

Field observations have shown that joints within a rock mass frequently display preferential directions or sets of joints. These joint sets will generally possess similar characteristics between individual joints within the set. However, each set within the rock mass will possess its own unique characteristics. A large variation in the spacing, infilling materials, and physical characteristics of the surfaces of a joint set or faults, is common. Therefore, one discontinuity may possess shear strengths and frictional characteristics different from those of another set. The various properties should be assessed individually.

There are numerous properties of structural discontinuities that require evaluation at the investigation stage of rock slope stability. They include orientation and position in space, continuity, infilling characteristics and aperture, spacing, asperities, previous shear movement, rock type, and hardness. These properties are discussed briefly below:

Orientation: The orientation, such as dip/dip direction of the discontinuity, is probably the most important property. The potential instability of a slope is proportional to the orientation of the discontinuity as it approaches the orientation of the slope and the dip angle as it approaches or exceeds the angle of friction along the discontinuity.

Continuity: Continuity is generally recorded as the observed linear length. The average continuity of a set defines the magnitude or size of the potential failure. Also, the effective shear strength on a failure surface that comprises one or more discontinuities is a function of their continuity.

Infilling: Infilling comprises the material located between the walls of the discontinuities. The most important characteristics of infilling, are its thickness, type, strength, and dissolution potential.

Aperture: Aperture is a measure of the separation between joint surfaces. Shear strength along the joint generally increases with the surface contact area.

Spacing: Spacing is a measure of distance between the joints and is defined as the distance measured perpendicular to the trend of the joints. A rock mass with a close joint spacing is weaker than the same mass with a larger spacing.

Figure 2-1. Foliation in metamorphic phyllite schist. Toppling failure developing.

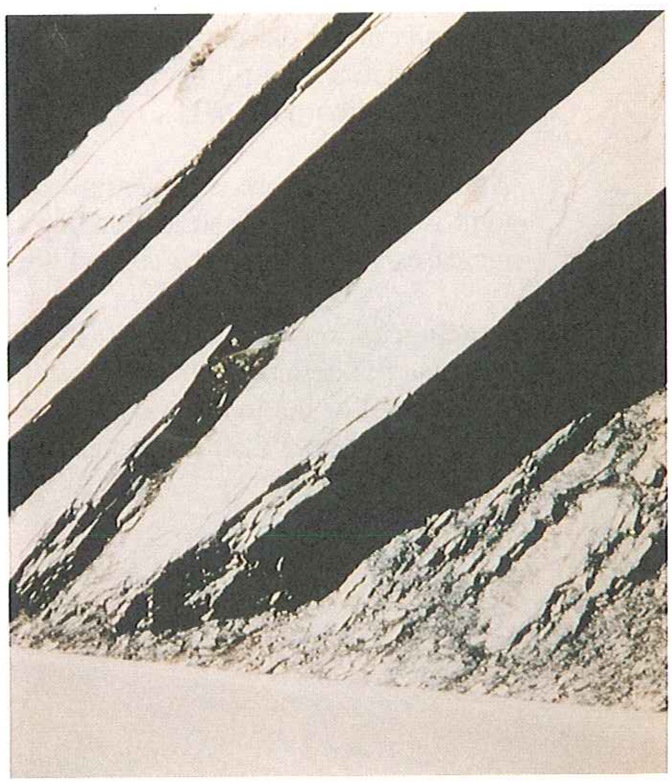


Figure 2-2. Bedding discontinuities in a slate. The slope was drilled at 1/4:1 angle with closely spaced drill holes. However the slope failed along the bedding at 55° when the rock was excavated. The overbreak should have been expected.

Figure 2-3. Randomly jointed rock. Steep slopes generally can be developed provided careful blasting is employed. Jointing is frequently developed in three perpendicular directions.

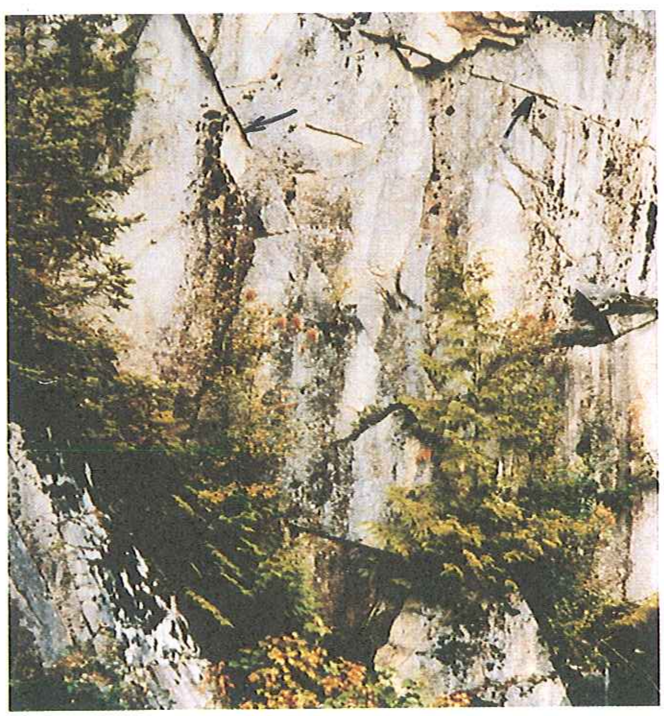


Figure 2-4. Throughgoing joints in hard granite. Blasting and stress relief can cause movement along the joints and result in failure.

Figure 2-5. A wide fault zone in hard granite. If this zone dipped out of the slope or was parallel to the slope, instability could occur. The shear strength in the fault would control stability.

Asperities: Asperities consist of two types: the surface waviness and surface roughness. Undulations or waves of the discontinuity can reduce the apparent dip of the plane and adjust the direction of motion of the plane during failure. Surface roughness usually consists of many small asperities that generally shear off during movement. These asperities can produce an increase in the in situ angle of friction related to the average angle of incidence (figure 2-6) relative to the average angle of the plane. Rough asperities can increase the effective friction angle by as much as 15 to 20° (Figures 2-7, 2-8). Note the surface roughness likely will vary in all directions. The roughness effect is most important in the downslope direction of movement.

Rock Type: Differing rock types within a slope can produce variations in rock and discontinuity strengths. The properties of each rock type will influence significantly the friction angle, properties of the asperities, and hardness of the joint wall rock.

Hardness: As a general rule, the shear strength of any discontinuity is a function of the shear strength of infilling material and hardness of the surface asperities. An increase in the rock hardness will increase the shear strength.

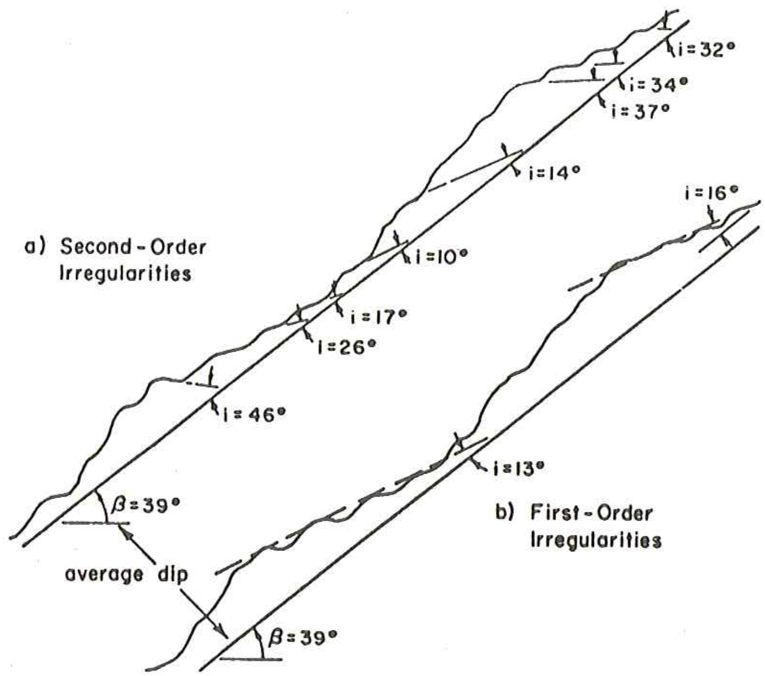


Figure 2-6. Surface asperities that increase the effective angle of friction along discontinuities. The orientation and magnitude of these asperities differ in direction. Evaluation in the direction of sliding is essential (Patton, 1966).

A typical discontinuity mapping sheet is shown in figure 2-9 (FHWA, Rock Slopes, 1989). Most organizations will develop such a sheet to their requirements.

2.3. LITHOLOGY

Highway rock cuts generally will be comprised of different rock types with various grades of weathering and alteration. Knowledge of the different lithologies and the degrees to which weathering and alteration have occurred will provide a basis for estimating how the rock will behave. Each lithologic type will have its own unique properties, such as macro and micro structural discontinuities, mineral assemblage, texture, and strength (table 2-1).

Figure 2-7. Micro asperities that will have a major influence on the in situ joint shear strength. Note the asperities have different roughness in each direction that will influence the frictional change as much as 15 to 20°.

Figure 2-8. Large-scale macro roughness that can influence in situ shear strength. The board is 12 feet (3.7 meters) long.

The intact rock strength can have significant impact on the shear strength of the rock, especially if there is an absence of infilling material. The intact strength of the asperities will dominate the resistance along the joint surface because of the shearing that must take place. The frictional resistance of the joint surfaces are dependant upon the percentage of the various minerals that are exposed.

Weathering also alters the strength characteristics of the rock. Atmospheric elements and conditions commonly cause weathering. It can reduce the intact rock strength by degrading minerals to secondary products, such as feldspars to a kaolinite. Weathering also is dependant upon climatic conditions, with the depth of weathering generally extending much deeper in a tropical climate than that of a temperate climate. There is a substantial gradient to the depth of weathering north-south on this continent. Generally glaciated areas have shallow depths of weathering. Weathering of interbeds may also occur, such as between basalt flows. Weathering of interbeds in slopes results in undercutting competent beds, followed by failure (figure 2-10). Where interbedded materials occur and one material is weak, block-type failures are common (figure 2-11).

When some rock types such as those that contain montmorillinite are exposed to the atmosphere for the first time, the mositure in the atmosphere can cause swelling, slaking, and differential movement of the rock mass. High swelling pressures are induced in the rock as more and more moisture is absorbed. Montmorillinite is also a secondary mineral and may occur in joint infilling materials, as will graphite, talc, calcite, and chlorite. Because all these layer lattice-type minerals possess a low shear strength, their presence must be determined.

Stress relief causes rebound when rock is excavated. Where different rock types are in contact, the rebound will be differential and will cause shear strain along the contact. This may result in strength loss and instability.

STRUCTURAL MAPPING CODING FORM

1. Location, orientation, and number of planes are numeric: data unit is alphabetic and/or numeric; all others are alphabetic.
2. Surface type, line type and rock type are three letter mnemonics; infilling, water, form, roughness, and termination are one letter mnemonics.
3. Record all mnemonics and their full proper descriptions on a reference sheet.
4. Location records position within the data unit or traverse, each data unit should include data from within one structural unit only.
5. Thickness, spacing, and length are entered according to the size notation given below.

Traverse (Data Unit) Information

DATA UNIT | NORTHING | EASTING | ELEVATION | INCLINATION

BEARING | LENGTH | NO. POINTS | STRUCTURAL UNIT | FORMATION | DECLINATION | OBSERVER

REMARKS:

SIZE NOTATIONS:

A	< .01"	E	.5–1 "	J	1–2'	O	30– 60'
B	.01–0.1"	F	1–2 "	K	2–4'	P	60–100'
C	0.1–.25"	G	2–4 "	L	4–8'	Q	100–200'
D	.25–0.5"	H	4–8 "	M	.8–15'	R	200–400'
		I	8–12"	N	15–30'	S	> 400'

DATA UNIT							JOB NUMBER			
LOCATION	Surface Type	Orientation Dip	Orientation Direction	INFILLINGS 1 2 3	Thick	Water	Form	Rough	Spec	No. of Planes	Length	Term	Line Type	Orientation Dip	Orientation Direction	ROCK TYPE

Figure 2-9. Typical discontinuity mapping sheet (FWHA *Rock Slopes* 1989).

Table 2-1.

Approximate Classification of Cohesive Soil and Rock (Robertson, 1987).

NO.	Description	Uniaxial lb/in^2	compressive kg/cm^2	strength MPa	Examples
S1	VERY SOFT SOIL - easily moulded with fingers, shows distinct heel marks	<5	<0.4	0.04	
S2	SOFT SOIL - moulds with strong kneading	5-10	0.4-0.8	0.04-0.08	
S3	FIRM SOIL - very difficult to mould with fingers, indented with finger nail, difficult to cut with hand spade	10-20	0.8-1.5	0.08-0.15	
S4	STIFF SOIL - cannot be moulded with fingers, cannot be cut with hand spade, requires hand picking for excavation	20-80	1.5-6.0	0.15-0.60	
S5	VERY STIFF SOIL - very tough, difficult to move with hand pick, pneumatic spade required for excavation	80-150	6-10	0.6-1.0	
R1	VERY WEAK ROCK - crumbles under sharp blows with geological pick point, can be cut with pocket knife	150-3500	10-250	1-25	Chalk, rocksalt
R2	MODERATELY WEAK ROCK - shallow cuts or scraping with pocket knife with difficulty, pick point indents deeply with firm blow	3500-7500	250-500	25-50	Coal, schist, siltstone
R3	MODERATELY STRONG ROCK - knife cannot be used to scrape or peel surface, shallow indentations under firm blow from pick point	7500-15000	500-1000	50-100	Sandstone, slate
R4	STRONG ROCK - hand-held sample breaks with one firm blow from hammer end of geological pick	15000-30000	1000-2000	100-200	Marble, granite, gneiss
R5	VERY STRONG ROCK - requires many blows from geological pick to break intact sample	>30000	>2000	>200	Quartzite, dolerite, gabbro, basalt

Figure 2-10. Weathered interbed between basalt flows. The differential weathering undercuts competent rock above resulting in rockfalls. Erosion must be controlled to halt the weathering -such as by shotcreting.

Figure 2-11. Interbedded sandstone and shale formation. When the shale is weak, block type failures are common. Differential erosion of the weaker beds may also occur.

2.4. GROUNDWATER

Groundwater (surface water and subsurface water) can have a significant impact on the stability of rockfalls and rock slopes. The direct relationship between the degree of water infiltration into a slope and the decrease in the stability of the slope has been well-documented in the literature. A good understanding of the hydrogeology of the site, including seepage, pore pressure distribution, and the factors that affect them, is critical. Surface water flowing over the slope can erode around and below rocks and lead to rockfall.

Factors such as groundwater flow, hydraulic conductivity, recharge and storage will be a function of the geologic structure, stratigraphy and lithology. Variations in the climatic conditions resulting in fluctuations in the phreatic surface, recharge and pore pressure distribution must also be appraised.

Groundwater can affect rockfall and rock slope stability in the following ways:

- Reduction in the frictional shear strength due to hydrostatic uplift in discontinuities, which reduces the effective normal stress.

$S = N \tan \phi$ becomes $S = (N-\mu) \tan \phi$

where S = shear strength
N = normal weight of rock above the discontinuity
μ = pore water pressure
ϕ = angle of friction

- Reduction in the cohesive strength of clayey-type rocks.

- Creation of seepage forces due to water flowing through the rock toward the slope face.

- Development of water pressure in joints and tension cracks. This pressure increases proportional to the square of the crack depth and is independent of the crack width.

- Hydrodynamic shock forces due to blasting below the water table.

- Ice jacking, which develops when water freezes in the cracks.

- Base exchange caused by seepage flow that carries different ions than exist in the natural rock and faults, for example, a sodium clay being changed to a calcium clay.

- Oxidation due to fluctuating water conditions that results in expansion of minerals.

- Dissolution of various mineral types, such as gypsum, limestone, dolomite, or rock salt.

2.5. CLIMATE

Several climatic conditions contribute to the instability of a rock or rock slope. Temperature variations, rain, snow, freeze-thaw and erosion conditions can act independently or in conjunction to cause stability problems. Fluctuations in the groundwater position from seasonal rainfall, runoff, or ice on the slope face can induce a wide range of hydrostatic pressures in the slope, potentially enough to cause small failures along pre-existing cracks.

Where temperate climates exist, freeze-thaw cycles commonly are the cause of rockfalls. When temperatures are above freezing, the frequency of rockfalls is proportional to the amount of rainfall (figure 2-12). During periods when the mean temperature is near 32°F (0°C), the frequency of rockfalls increases because of the frequent freeze-thaw cycles that occur (figure 2-13).

Frost action also can contribute to rockfalls. Since water undergoes an approximate 9 percent volume increase when it freezes, the volume expansion can create large pressures in a confined space, such as cracks or joints. These become more severe in colder climates and are commonly referred to as ice wedging or jacking.

Where trees exist on the slope or at the crest, and the roots have developed into the discontinuities, tree-root leverage is a common cause of rockfall. High winds acting on isolated trees can lever movement of large rocks (Figure 2-14, 2-15).

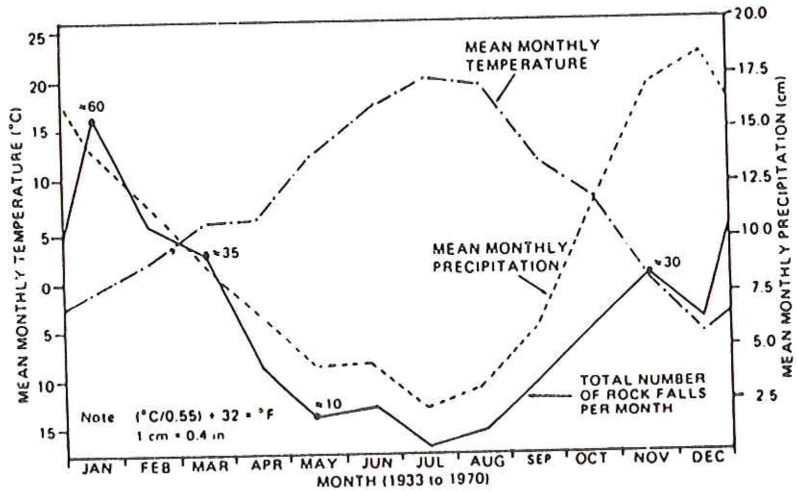


Figure 2-12. Correlation of the number of rockfalls with temperature and precipitation on railway lines in the Fraser Canyon, British Columbia (Peckover, 1975).

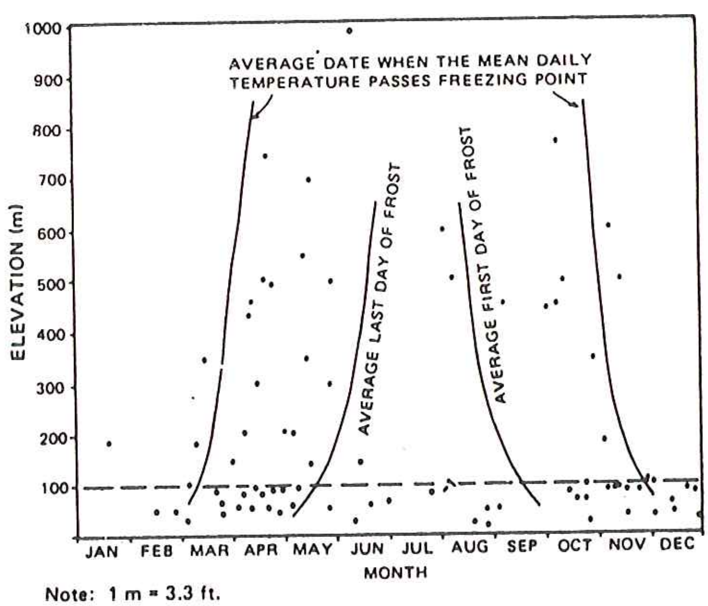


Figure 2-13. Rockfalls in eastern Norway in relation to altitude, time of year, and temperature. Dots indicate rockfalls occurring during different seasons (Piteau, 1977).

Figure 2-14. Tree with roots growing into a joint. Note that the joint has been pried open several inches.

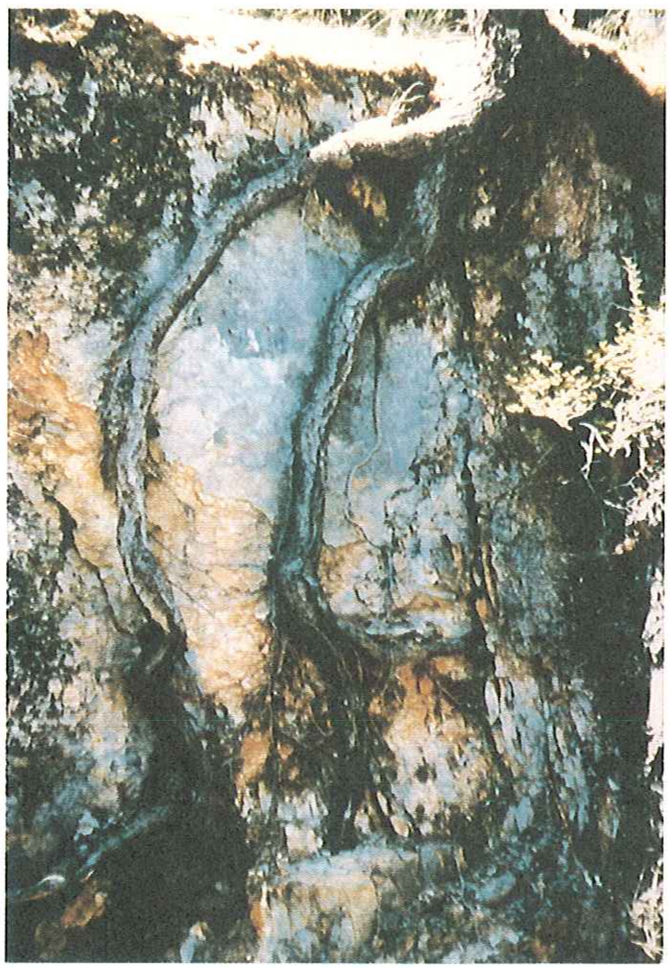


Figure 2-15. Tree roots that have pried a rock loose and created a rockfall. Winds can create high leverage forces and cause numerous falls. Trees on slope and within about 6 feet of slope crest should be removed.

2.6. NATURAL AND EXCAVATED SLOPES

Over time, natural rock slopes are exposed to forces that lead to deterioration of the rock mass. These forces are generally dependant upon the effects of regional stresses, physical and chemical weathering, surface erosion, rainfall variations, freeze-thaw cycles, and temperature fluctuations.

This results in the relaxation of the rock mass by opening fissures and joints, and a zone of rock to a certain depth becomes progressively weaker with time. This type of deformation is irreversible, and movements cause displacements along joint surfaces in the rock mass that inherently reduce the shear strength and thus the overall strength of the rock mass.

Excavated slopes also deteriorate with time. After excavation, the slope will gradually weather and adjust itself toward equilibrium. Near-surface failures, such as rockfalls and ravelling, generally develop after only a few years following excavation. Some types of rocks, such as shale or mudstone and soft argillaceous rocks, can undergo deformations and slaking in significantly short periods of time, sometimes within a few weeks.

Stresses within the rock mass are now known to be more complex than previously realized. Rocks are not only subjected to vertical forces due to their weight, but also can be subjected to horizontal stresses caused by tectonic forces, deep surface erosion, or glacioisostatic or excavation rebound. High horizontal stresses can cause differential rebound in adjacent rock of different lithology and cause softer rock masses to deform after excavation. This decreases the shear strength along discontinuities and increases their susceptibility to weathering and alteration.

Where there is a choice of construction in the route selection along the north side of a valley as compared to the south side, the variation in freeze-thaw cycles, hot-cold temperatures, seepage and snow should be considered.

2.7 BLASTING AND VIBRATIONS

The majority of highways in the United States were developed before controlled blasting procedures were specified and used. Today, most of these rock cuts experience instabilities as a result of the limited blasting control.

Blast damage of a rock slope can result in severe maintenance problems over the long term. Poor blasting techniques result in overbreak, extensive shattering of the rock slope, and development of tension cracks in the slope. The migration of gases along pre-existing structure or new cracks developed during the tensile fracture process can propagate well behind the face and allow more infiltration of water into the slope, which creates excess pore water pressures, greater frost susceptibility, and increased weathering.

The relationship of blasting energy expressed as particle velocity vs distance to the final slope face vs charge weight per delay, is shown in figure 2-16.

Minimum rock damage can be obtained if the blast designer carefully selects the blast hole size and pattern, explosive type and load, and blast delay sequence. Blasting patterns should be determined based on empirical experience and test blasts in the field. Review of blasting records in similar areas, or the evaluation of a blast design while changing the design parameters should be developed from the test blasts. Reference to the Federal Highways Administration publication, Rock Blasting and Overbreak Control (1991), is recommended.

Vibrations also cause damage of the rock face when care is not exercised near the final wall. It is essential to protect final wall from the production blast by using buffers and cushions or presplit blasting with delayed detonation. These techniques minimize the energy impacted to the final wall and can greatly reduce long-term maintenance problems. Figures 2-17 and 2-18 show the results of control blasting and uncontrolled blasting.

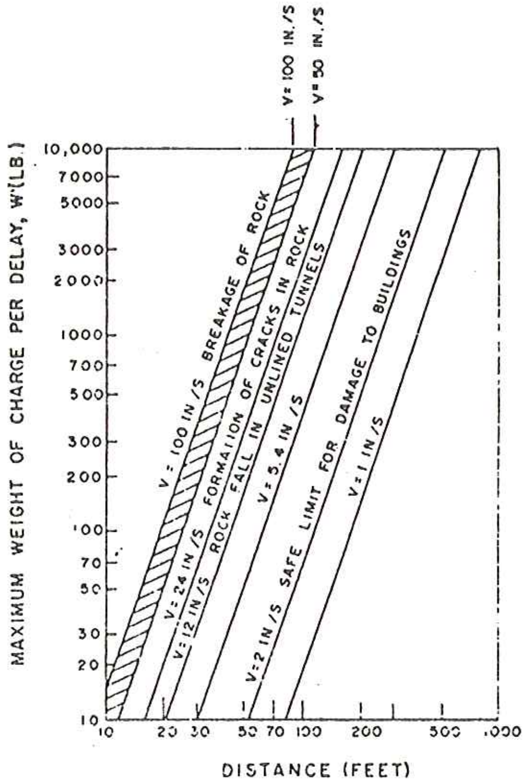


Figure 2-16. Variation of maximum particle velocity with distance and weight of charge per delay (average rock conditions) (Brawner, 1974).

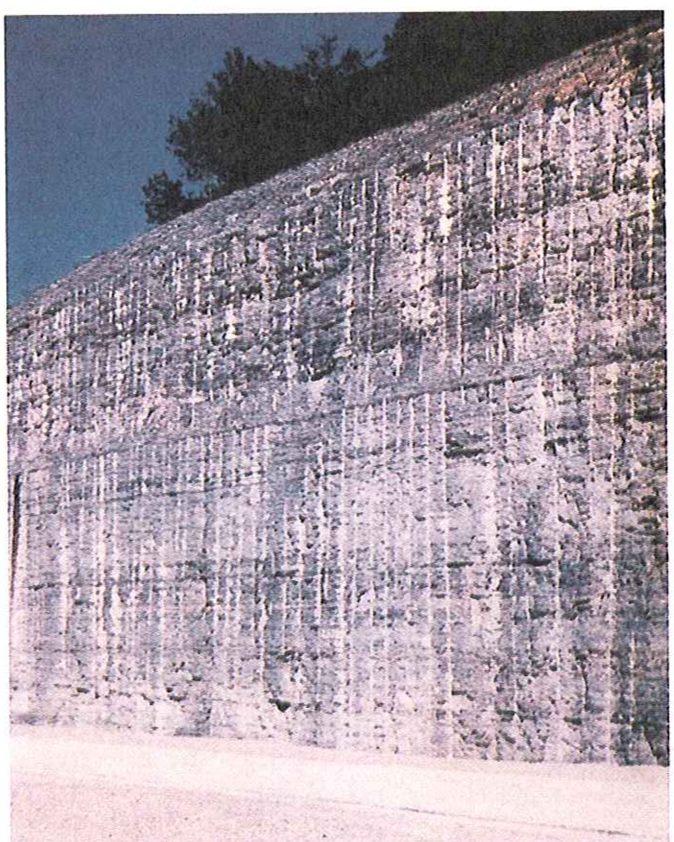


Figure 2-17. Very closely controlled presplit blasting to develop a very uniform vertical rock slope.

Figure 2-18. Very hard granite gneiss severely damaged by excessive blasting forces. This slope will be subject to unnecessary long-term ravelling.

The development and implementation of controlled blasting to minimize damage to rock slopes has increased the safety along today's highways, reduced maintenance costs due to reduced rockfall and ditch cleanup, and reduced environmental impact by allowing use of steeper cut faces.

Vibrations from blasting and from equipment such as trains, construction equipment, or heavy trucks can induce rockfalls in well-jointed rock masses. The nearby passage of long unit trains also cause harmonic vibrations that cause rockfalls.

Dynamic forces and vibrations due to earthquakes have become an important design requirement of rock slopes, especially along the western coast on North America. Long-period vibrations with prolonged durations during an earthquake can cause excessive vibrations of a rock slope and may cause excessive pore water pressures and local rock ravelling. Scaling and regular maintenance, as well as adequate slope drainage, are effective means of reducing the possibility of rockfall.

2.8. NATURAL PROCESSES

Erosion of the slope by the natural processes of weathering, precipitation, and vegetation growth, and erosion by humans and animals also can cause rockfalls. This is an ongoing problem on talus or glacial till slopes with large boulders.

Weathering primarily reduces the overall rock mass strength, decreasing the ability for an existing steep slope surface to support itself, which results in rockfalls. Erosion by precipitation and runoff can remove interstitial soil and infilling materials from the structural discontinuities and weathered rock and expose a greater area for water and frost penetration.

Vegetation growth is generally known for its stabilizing qualities in soil slopes. However, roots of larger trees located in rock discontinuities can cause joint apertures to increase over time, reducing the frictional resistance and increasing the susceptibility of the rock mass to precipitation and frost action. Wind leverage forces on trees on the slope and the rock crest cause many rockfalls.

Animals, such as deer, elk, mountain sheep, and mountain goat frequent rock slopes and occasionally cause rockfalls. Human hikers also may dislodge rock on steep slopes.

CHAPTER 3

TYPES OF FAILURES

The importance and influence of structural geology on rockfall stability has been described in chapter 2. Most types of failure will be influenced by the dip, orientation, and extent of the structural discontinuities. The principal exception is the circular-type failure.

One of the major reasons to establish the type of failure mode is that the method of stabilization that applies to one mode may not be effective for another mode. For example, unloading the top of a circular failure will improve the stability but unloading a planar or wedge failure will not.

Various types of failure are described in the following text with typical examples included.

3.1. CIRCULAR FAILURE

Circular failure is generally associated with rock that has low strength and low-to-moderate variation in strength. Weak failure zones with strength less than the rock mass are not oriented to control stability, rather the failure is controlled by the low strength of the rock mass. This type of failure is most common in the non glaciated areas of the United States, particularly in the south.

Typical conditions include weathered rock or weak sedimentary rock where bedding does not dip out of the slope. Typical examples of failure are shown in figures 3-1 and 3-2.

In shallow cuts, the slopes may be entirely in weak rock. However, there will be many cases where weak or weathered rock overlies hard stable rock and the failure will develop high on the slope.

Since this type of failure is similar to circular failure in soil, soil mechanics analyses and principles apply. The failure is characterized by a drop in the crest and a push out at the toe. No strongly defined structural pattern exists so the failure zone is free to find the line of least resistance through the slope. The shear strength of the material is characterized by frictional and cohesive strength parameters.

Figure 3-1. Circular failure in weak weathered rock. The failure filled the ditch and encroached on the highway shoulder. The toe of the movement was excavated. Heavy rainfall can cause the zone to move again. The top should have been unloaded to counterbalance the toe excavation.

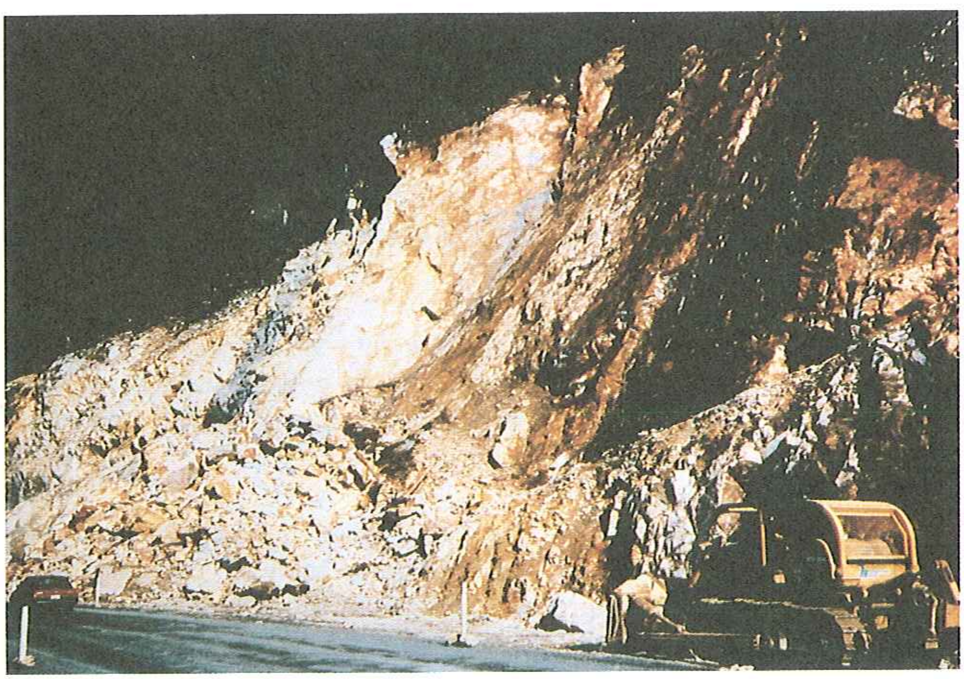


Figure 3-2. Circular rockfall in weak weathered rock. This fall occurred during construction due to oversteepening of the slope.

In the majority of cases, a tension crack will develop at the top of the failure. Where the volume is small, the crack will be narrow. If surface water gets into this crack, the water pressure may be sufficient to cause failure. For small volumes, the failure usually will be rapid.

The majority of circular failures are caused by oversteepening the slope or by an increase in pore water pressure in the slope.

Common stabilization procedures for small circular failures include total removal of the slide, unload the top, load the toe, or drain the slope to reduce pore water pressures.

3.2. PLANAR FAILURE

Planar failure occurs where rock slides on one continuous discontinuity which dips out of the slope. Usually the slope has been undercut. Photographs of typical examples are shown in figures 3-3 and 3-4.

The following conditions must be satisfied in order for sliding to occur:

- The slide plane must be nearly parallel to the slope face. Hoek and Bray (1981) suggest a variation of not more than $\pm 20°$ ($\pm 68°c$).

- The failure plane must daylight on the slope face.

- The dip angle of the failure plane must be greater than the effective angle of friction for the slope with no water pressure.

- Release surfaces or planes with negligible shear resistance must exist at the lateral boundaries of the mass.

It is obvious that planar failure will be more common with steep slopes, at the location of rock noses on curves, and in areas of high precipitation and seepage. Where steep slopes exist, the shear stresses can be high enough to shear "rock bridges" between bedding planes.

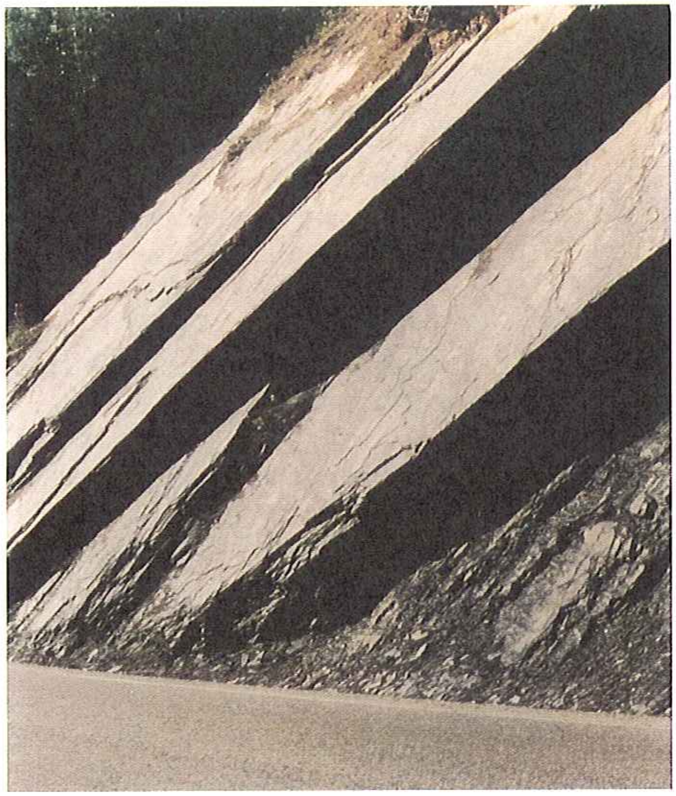


Figure 3-3. Planar failure in bedded slates. Dip angle about 55°. The strike of the bedding is about 15° off the strike of the centerline. Note the failures daylight in the ditch. These falls occurred when the blasted rock was excavated during construction.

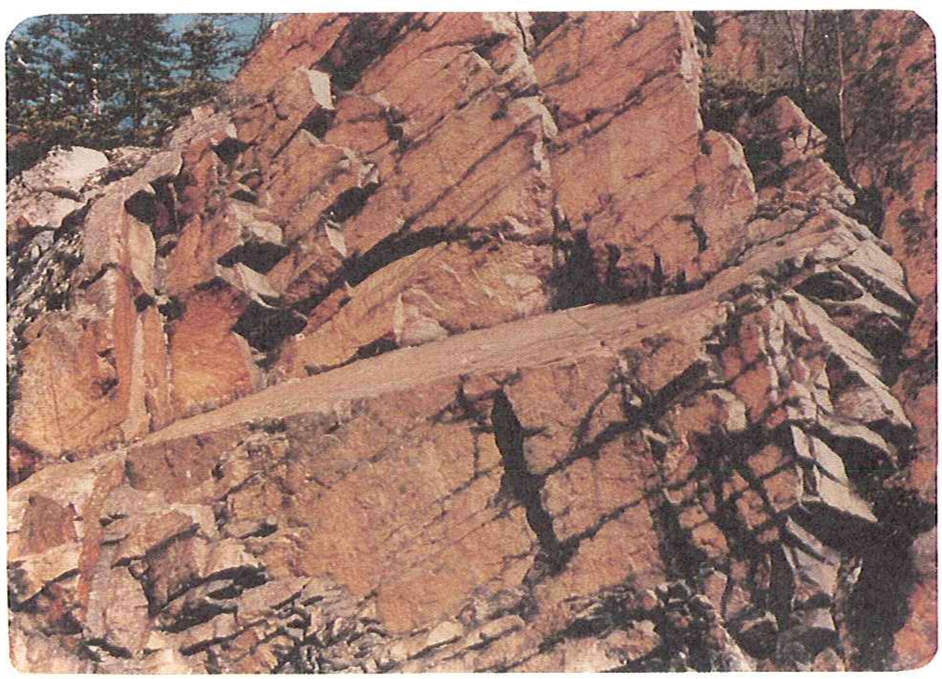


Figure 3-4. Planar failure on a steep through-going joint. The back of the block is bounded by steep dipping joints. This fall fell onto the highway.

A tension crack may develop at the top of the slope prior to failure. At this time, the stability is low. Small scale planar slides or falls usually occur rapidly.

The magnitude of blasting seismic forces developed during original slope construction will impact the potential for planar movement. Large blasts open up discontinuities and develop new cracks. Blasting also may cause some of the asperities along the failure surface to be sheared to reduce the effective friction along the discontinuity.

Water pressure can develop in any tension crack and along the potential failure plane during heavy precipitation or snow melt periods. This can precipitate the failure.

Where many planar failures occur over a range of dipping discontinuity angles, measurement of these angles in the field will frequently indicate a minimum dip angle from which the falls slide. The lower angle is used to estimate the effective angle of friction along the discontinuities.

The majority of planar failures are caused by excessive blasting forces during rock excavation, oversteep slopes, or water pressures in the slope.

Common stabilization procedures include complete removal (preferable during original construction) drainage, doweling, and rock bolting.

3.3. WEDGE FAILURE

Where two or more discontinuities intersect, a rock wedge may develop. Photographs of typical wedge failures are shown in figures 3-5 and 3-6.

If the angle of intersection of the discontinuities dips out of the slope at an angle steeper than the angle of friction along the surface with lowest shear resistance, the stability must be suspect. In addition to this affect, the angle between the wedges must be considered. The safety factor increases as the confining angle reduces. Where water pressures develop in tension cracks or in the discontinuities, the safety factor will be reduced substantially.

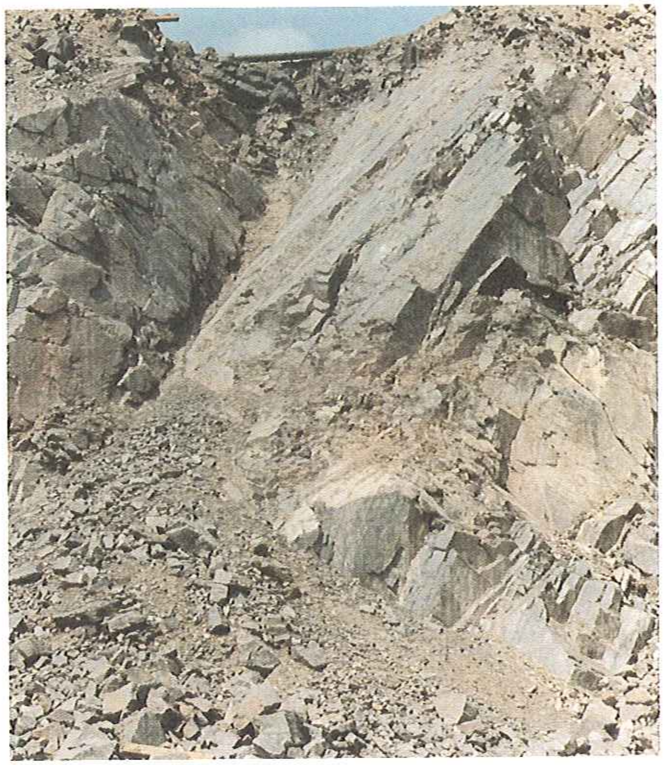


Figure 3-5. Wedge failure with angle of intersection steeper than the angle of friction along the rock discontinuities.

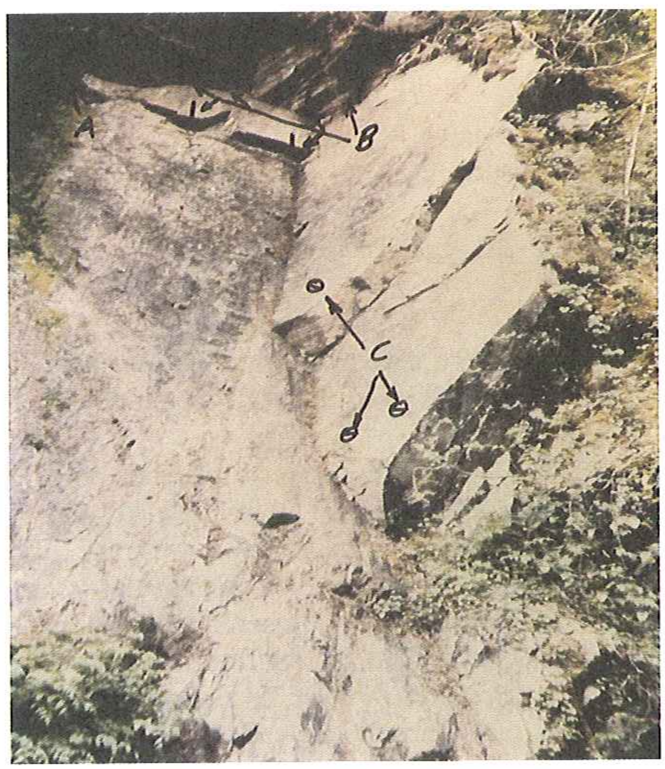


Figure 3-6. Wedge failure on steeply dipping joint surfaces (example A). To control an extension of the failure, dowels were recommended at B and tensioned, grouted bolts at C.

Where the volume of the wedge is small, failure can occur rapidly. The majority of wedge failures will occur during excavation of the rock as the toe of the wedge is undercut. Any wedges identified during road construction that have an intersection dip angle that exceeds the estimated shear strength along either discontinuity should be excavated or stabilized during construction.

The majority of wedge failures are caused by oversteepening the slope, use of excessive blasting energy during construction, or water pressures that develop within the slope. Wedge failures that happen after construction usually occur rapidly and frequently during heavy precipitation or snow melt events.

Common stabilization procedures include complete removal, slope drainage, or rock bolting.

3.4. BLOCK FAILURE

Where reasonably flat discontinuities with low strength exist, block-type movements can occur. Photographs of typical block failure conditions are shown in figures 3-7 and 3-8. Block movements are most common in sedimentary rock, with layers of clayey rocks (shale, slate, mudstone) containing smectite, bentonite, or montmorillinite swelling minerals. With excavation, lateral stress relief occurs. Where layers with different rebound moduli exist, differential strain occurs at the contact, which reduces the shear strength along the contact. In addition, the lateral movement tends to open joints or develop steep tension cracks in the slope or beyond the crest. Subsequent precipitation or snow melt that results in water pressure in the open crack can develop sufficient lateral force to cause the block to move.

If the difference between the peak strength and residual strength along the discontinuity is large, the block can fail completely and slide down the slope. If the crack behind the block fills with water, the water pressure can actually push the block out on a slightly upward slope to about 5°.

Block failures are more likely to occur with flatter slopes, since the normal load that increases frictional resistance is reduced while the slide surface area remains the same.

Figure 3-7. Block failure that occurred on a near-horizontal shale layer in bedded sandstone. Lateral stress relief opened up tension cracks in the slope. Water pressure developed in the crack during heavy precipitation and pushed the blocks off.

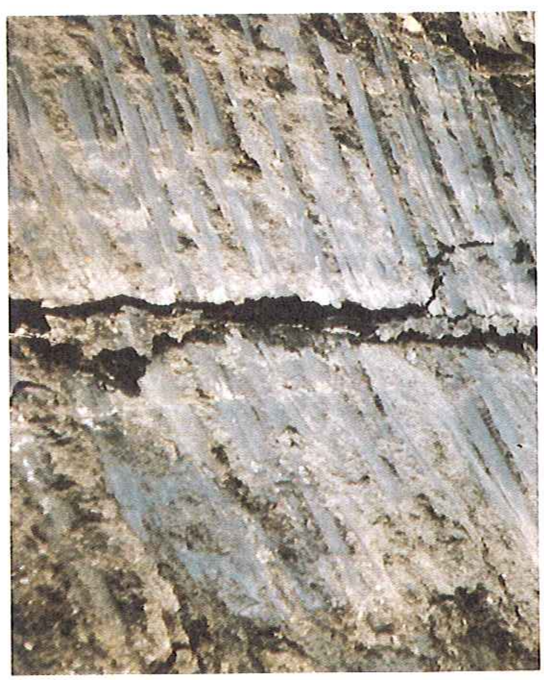


Figure 3-8. A block failure developing along a flat weak clay layer in a weak, dark sandstone.

Block failures require the existence of nearly flat weak layers and water pressures to develop in a back tension crack or joint.

Stabilization involves the control of surface water entering the crack. Steepening the face, which increases the normal load, will improve stability provided the slope face will remain stable. Horizontal drains may also provide drainage.

3.5. TOPPLING FAILURE

Where parallel, closely spaced discontinuities dip steeply and are parallel or near parallel to the slope face, the rock may topple toward the roadway. Generally, the discontinuities will dip into the slope in excess of about 50°. On occasion, toppling has occurred where the discontinuities dip steeply out of the slope.

Toppling occurs most frequently in bedded sediments and columnar basalts excavated with a steep face angle. Typical examples of toppling are shown in figures 3-9 to 3-11.

Toppling is aggravated by water pressure and ice pressure in the winter, which develops in the steep dipping discontinuities. As movement continues, rock breaks up on the face and ravelling occurs. Where the discontinuities dip near vertical, entire outer slabs may topple. If the toppling column height exceeds the ditch and shoulder width, the rock can topple onto the roadway.

Ravelling that results from toppling is generally a slow and ongoing process. However, slab toppling can be rapid. In volcanic columns or basalts, the columns may topple when the weak interbed material is eroded from under the columns.

Toppling failures occur because of adverse geologic structure, the use of steep slopes, water pressure in tension cracks (which may be multiple) and erosion, weathering, or overblasting at the toe.

Stabilization generally involves flattening the slope, bolting near the crest of the slope, controlling surface water, and draining the slope with drain holes. Ditch design is important to catch ravelling rock.

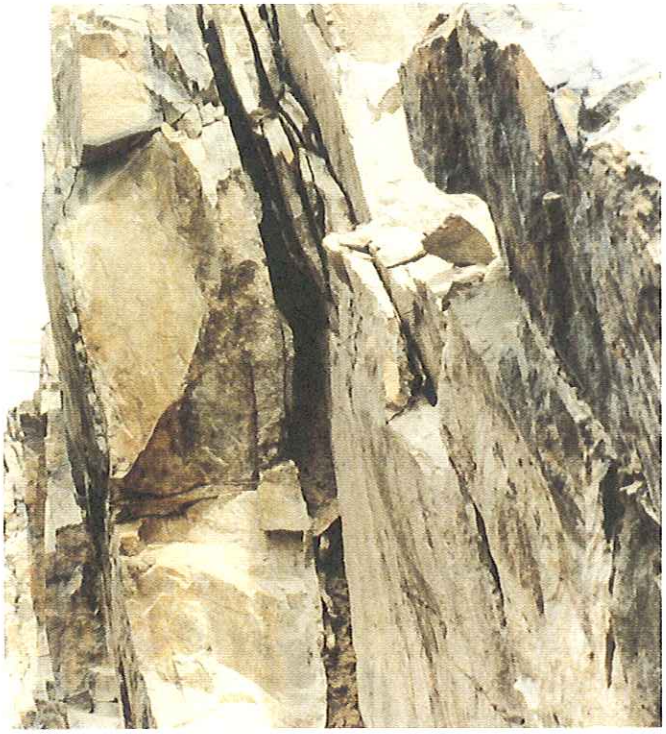


Figure 3-9. Toppling failure developing in hard bedded sandstone. Bed thickness varies. The slope was excavated very steeply so that slabs fell onto the highway.

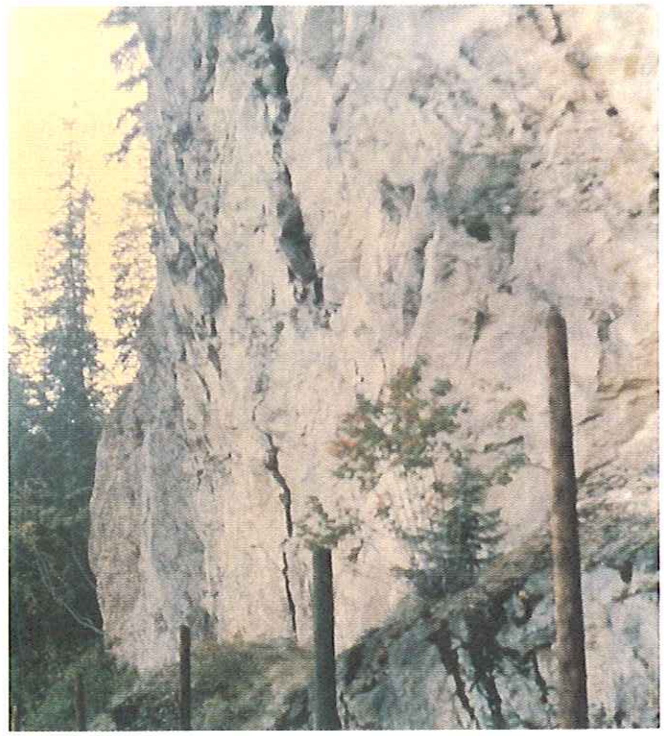


Figure 3-10. Large slab beginning to topple The volume was large enough that it could completely block the highway to traffic. The toppling slab was excavated during a controlled road closure.

Figure 3-11. Toppling failure with structure dipping into the slope at 40°. Note ravelling developing at the surface. A rough tension basal failure zone is developing.

3.6. BUCKLING FAILURE

Buckling failure can occur where excavetion follows closely spaced bedding that dips out of the slope at an angle that exceeds the angle of friction along the bedding. Typical examples of buckling are shown in figures 3-12 and 3-13. Slope excavation generally follows a bedding surface. Several types of buckling can occur:

- The gravity load within a rock layer down the slope is sufficient to cause buckling of the rock face.
- High water pressure in the slope may cause the rock to pop out.
- Secondary structure may cut across the bedding and buckling may occur at this zone of weakness.
- A roll may occur in the bedding such that a zone of overstress can result in buckling.

When buckling failure occurs, the movement is usually rapid and the rock above the buckling zone frequently slides down the slope.

Buckling generally occurs where the excavation follows the bedding, the bedding is closely spaced, the rock is brittle, the slope height is moderate to high, and water pressures exist in the slope. Excessive seismic forces because of blasting will also reduce stability. Buckling failures generally occur with no warning.

Procedures to stabilize a zone of potential buckling include flattening or benching the slope, installation of rock bolts, and slope drainage with horizontal drains.

3.7. KEY BLOCK FAILURE

Where variable rock structural orientations occur, one to several rocks may create a key to support or hold rock in place at a higher elevation. Removing the supporting key rock or rocks may cause numerous rocks to fall from above. Typical examples are shown in figures 3-14 and 3-15.

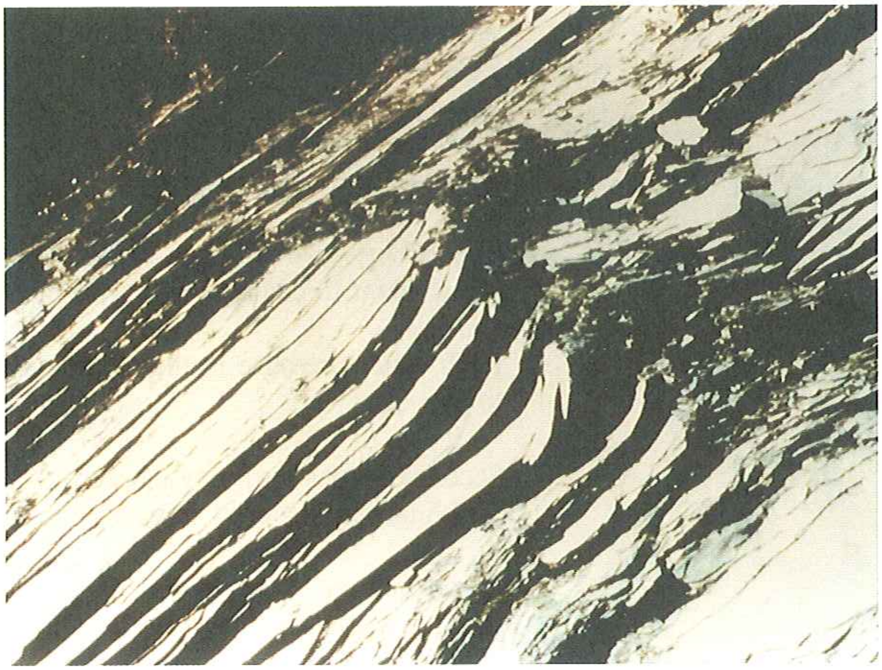


Figure 3-12. Buckling failure in thinly bedded shale. The failure was induced by water pressure in the bedding, which opened up due to lateral stress relief. Most of the upper slabs did not slide.

Figure 3-13. Buckling failure in bedded limestone at a fold in the structure. The upper slab failed.

Figure 3-14. Key block on steep joint supporting horizontally bedded rock. Cracks have opened, indicating the safety factor is low. The slope is only about 20 feet high (6 meters) so complete removal is warranted.

Figure 3-15. Key blocks have moved and the rock mass above is undergoing loss of support. The upper volume is very large. Rock bolting to support the upper rock mass is recommended. The key blocks should then be removed.

Removal of the key rock or rocks generally leads to rapid ravelling. The extent will depend on the volume of rock involved.

Key block conditions are usually related to rock structure and excessive blasting forces.

Where a key block situation is encountered during construction, it is usually best to remove all the rock that may be involved, particularly in areas of moderate-to-high earthquake potential. Where a large volume of rock exists above the key rock or rocks, bolting the key rock may be effective.

On existing highways, it may be cost effective and minimize traffic congestion to stabilize the key block by bolting or buttressing, rather than by removal.

Sketches of the various modes of failure are shown in figure 3-16.

3.8. RAVELLING FAILURE

Ravelling of one or several rocks from the slope may occur due to a multitude of causes. The danger develops when the ravelling rock has sufficient velocity and volume to reach the travelling surface.

Ravelling can be caused by water pressure behind the rock, ice-jacking, differential weathering or erosion along faults or shear zones, weakening of rock due to freeze-thaw, hot-cold or wet-dry cycles, and loosening caused by animals and trees. Ravelling also can result from vibration due to earthquake, construction equipment, or the nearby passage of unit freight trains. One of the most frequent causes of ravelling is tree root prying on the slope or near the crest. Tree roots develop and grow in discontinuities in the rock. High winds blow trees and create high crowbar-type leverage forces in the discontinuity. Under repetitive conditions rocks may become loose and fall. As the roots grow, they exert forces in the discontinuity that also may cause a rockfall.

Evidence of past rockfall can be observed from freshly exposed faces on the rock slope and from indentations (finger prints) in the asphalt surface.

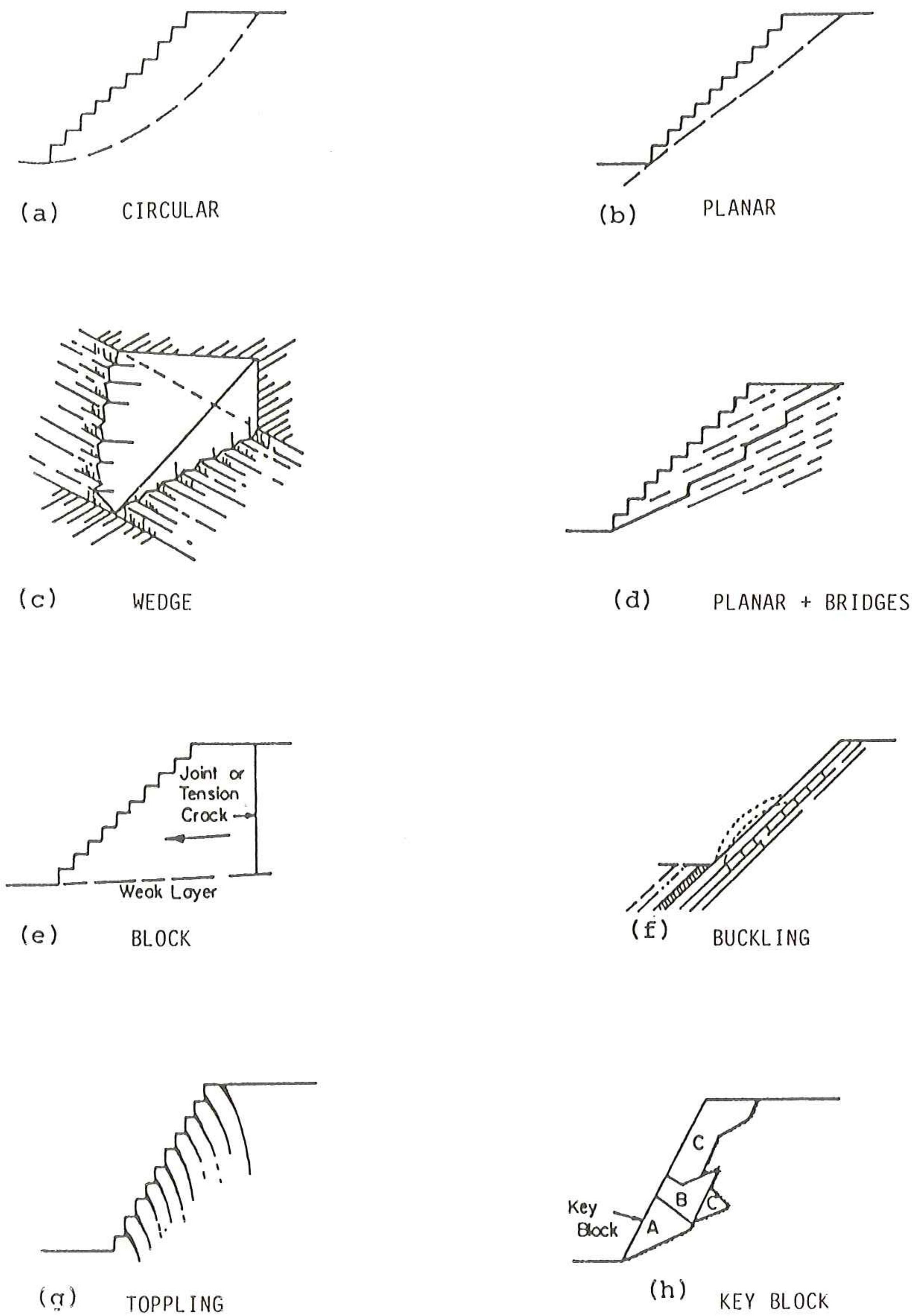


Figure 3-16. Sketches of various modes of failure.

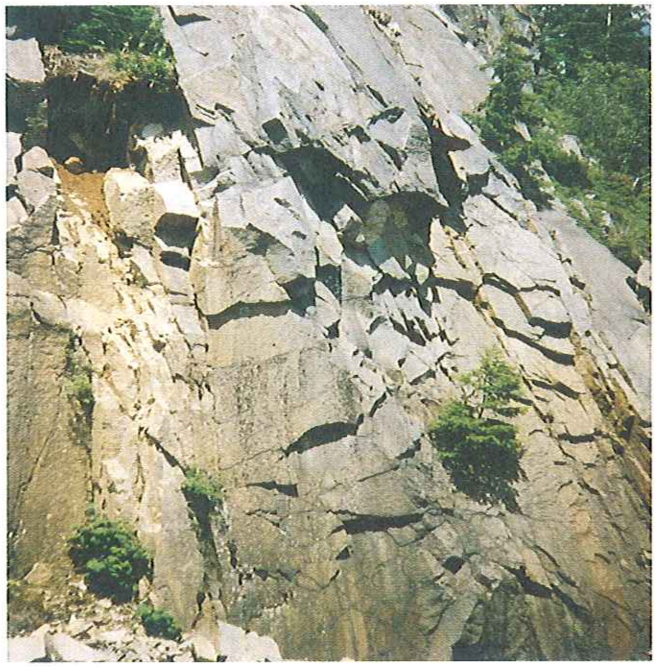


Figure 3-17. Ravelling rock along a steep fault zone. The rockfall occurs at various times. Freeze-thaw, hot-cold, and wet-dry cycles are major contributing factors.

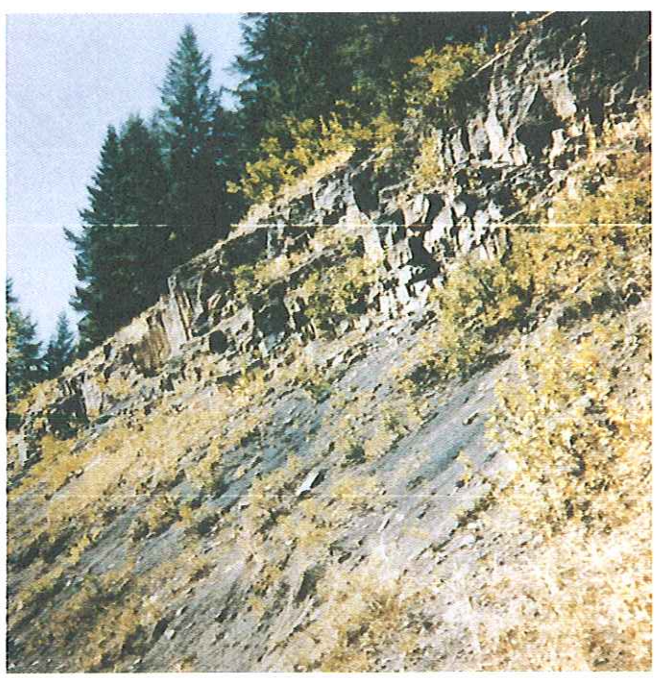


Figure 3-18. Blocky rock in the cliff face above the fine talus slope continues to ravel into the ditch.

Photographs of typical ravelling rock are shown in figures 3-17 and 3-18.

To reduce the potential of ravelling following new construction, all newly excavated slopes should be scaled by the contractor prior to leaving the site.

On existing highways, all trees in excess of about 4-inch diameter, which may lever rockfall, should be removed from the rock faces and slope crest back for about 6 feet.

Where there is potential for ravelling rock to reach the highway, periodic scaling of the slope may be considered. Alternatives include draped wire mesh and fences. If rockfall frequency increases, a geological and rock mechanics inspection, evaluation and stabilization program should be considered.

3.9. BOULDER FALL

On many highway projects the excavation will encounter colluvium, talus, glacial moraine, or till slopes, which contain large boulders. These boulders may range to more than 2 yd^3. The boulders usually are more rounded than rock from blasted excavations. As a result, they may develop more momentum and roll farther than angular rock. They will roll more readily through a ditch and they can be more dangerous to traffic than ravelling angular rocks.

In many cases, the boulder slope will overlie rock excavations. In this case the design should include a catch ditch at the soil-rock contact or mesh to control boulders.

Photographs of typical boulder slopes are shown in figures 3-19 and 3-20.

Ongoing surface water and wind erosion around the boulders results in their becoming exposed more and more until they roll or bounce down the slope. Freeze-thaw, wet-dry, and hot-cold cycles will also loosen boulders in the slope.

All boulders exposed more than about 40 percent of their surface during initial excavation and during later scaling should be removed. Larger boulders should be blasted smaller during the removal process to reduce potential roll-out damage. If the rock removal will leave an unstable hole, the outer portion of the rock should be removed with trim blasting or chemical expanders in drill holes. Caltrans only removes those rocks that can be removed by hand or with a pry bar.

4.0 DIFFERENTIAL EROSION

Where interlayered rock with variable strength or rate of weathering characteristics exist, the weaker rock tends to erode and undercut the more competent rock. This leads to revelling from the harder layers. Sedimentary rock and basalt with weak interlayers are common sources of differential erosion (figure 2-11).

Figure 3-19. Large boulders extruding from a dense glacial till slope. As erosion occurs around these boulders, they will roll down the slope. All such boulders should be removed or contained by a catchment ditch or barrier.

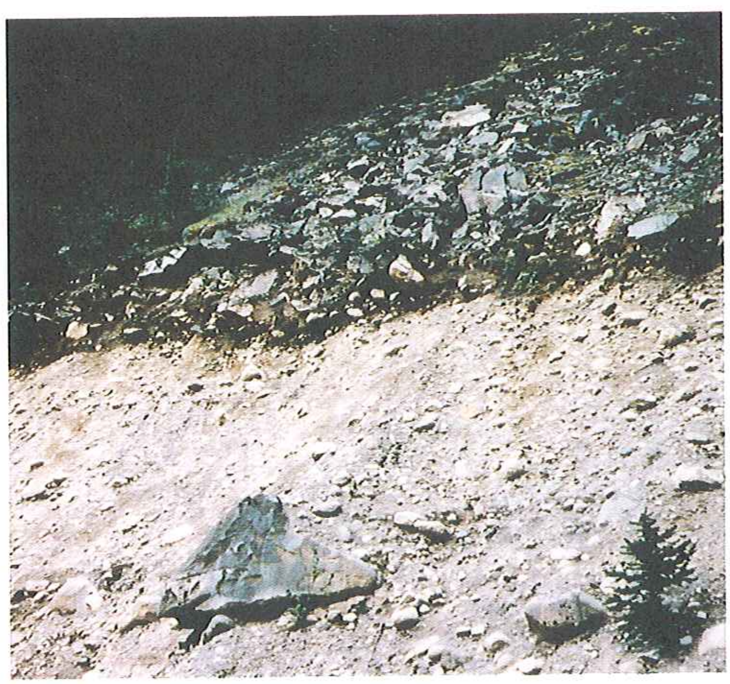


Figure 3-20. Undercut talus and colluvium slope exposing boulders that will roll down the slope. A catch wall is required in addition to a catch ditch. For high talus slopes, wire fencing or mesh is often used.

CHAPTER 4

SITE INVESTIGATIONS

4.1. INTRODUCTION

Site investigations are an integral part of the recognition and identification of potential rockfall locations, the type of rockfall, and determination of factors that contribute to the movements. Basic guidelines have developed through many years of experience in investigations of all types of rockfalls and rock slope failures. Guidelines for preliminary evaluation are summarized below:

- A review of topographic maps, geologic reports and maps, and engineering reports.

- Analysis of aerial photography and other forms of aerial images.

- Preliminary slope reconnaissance.

These investigations are usually followed by more detailed site-specific investigations. This helps to delineate the causes and determine the geologic, physical, and chemical properties of the rock conditions and topographic conditions so that proper priorities and design of remedial measures can be established. Economic considerations, traffic implications, legal ramifications, and a site rock stability priority rating assessment normally follow.

4.2. INVESTIGATION SEQUENCE

The sequence of the investigation techniques is important. Typical procedures have been developed and are as follows:

- Obtain aerial photographs and any other specialized imaging photographs.

- Review literature and information, such as geological and geotechnical reports, climatic data, design reports, construction records, highway geometry and sections, maintenance history, and traffic volumes and types.

- Interpret and analyze airphoto.

- Complete a final evaluation and plan further detailed investigations or develop designs and contracts for remedial measures.

- Perform an environmental evaluation and arrange for any necessary permits.

The following chapters discuss these procedures in more detail.

4.2.1. Literature Review

The review of existing reports from various agencies, such as the Geological Survey, generally can provide valuable geologic information in areas where no existing transportation routes exist. However, rockfalls are more commonly associated with existing highways. Four important aspects of the transportation route should be concentrated on during the literature review: the structural geology, climate, maintenance history, and traffic characteristics. The structural geology is discussed later.

4.2.2. Climate

The greatest number of rockfalls generally are associated with high rainfall, (figure 2-11) rapid snow melt, or worst of all, a combination. Water in cracks or discontinuities can cause high water pressures. Heavy snowfall may overload trees, which can cause them to topple and move rocks. Therefore, historic precipitation data must be obtained and reviewed. During these periods maintenance forces should be on special alert for rockfalls. Signs should be posted along the highway to alert the public of potential rockfall during wet periods.

Water freezing in cracks or discontinuities can gradually ice-jack rock loose. Hence, the period and seasonal number of freeze-thaw cycles should be determined. Figure 2-12 shows a relationship between freeze-thaw cycles and rockfall.

High winds exert severe leverage forces on roots in cracks. The occurrence and extent of windy periods should be evaluated.

The potential for erosion by surface water and wind erosion also should be assessed.

Large differential temperature changes that lead to thermal gradients in rock that are sufficient to break the rock in tension and lead to spalling. Southerly exposed rocks are more prone to this problem because of the greater range in temperature.

4.2.3. Maintenance History

The maintenance history of a particular slope or section of road can identify particular trends in rock slope failure, and most importantly, the rockfall occurrence and frequency. In most cases of smaller rockfalls, the failure mechanism can be attributed to a change in climatic conditions, such as precipitation, temperature, or wind. Graphic representations of rockfall history versus temperature, and precipitation or wind at or near the site can define these trends and identify the areas where further investigations are required.

An additional method of defining problem areas is to review accident history and accident causes. Unfortunately, because some States have only recently implemented the use of rockfall records, a good database on previous rockfall may not be available. It may therefore be worthwhile to research the accident history of a particular road section by interviewing traffic patrol officers and records and highway department maintenance personnel to identify areas of potential concern.

The size of the rockfall can indicate the potential slope hazard and indicate the need for certain types of preventative measures or support. Large, blocky rockfalls indicate structural control of the failure and therefore remedial measures such as bolts or dowels may be required. Smaller ravelling failures indicate remedial measures, such as scaling, shotcrete, installation of mesh or fences, or improved ditch design.

Greater attention must be placed on areas where a high frequency of rockfalls or rockfall occurrence is continual. These areas may be a precursor to larger rockfalls or slides. It is important that reconnaissance of the entire slope be made to identify the possibility of a much larger failure. For example, ravelling rock at locations 50 to 200 feet (15.3 to 61 meters) apart may be the early indication of a large movement between the ravelling.

In many areas, rock cliffs exist above talus slopes (figures 4-1 and 4-2). This requires that the upper steep slopes be inspected and assessed for rock instability. A single rockfall from a cliff face may loosen and create a small avalanche of rock from the talus slope below.

4.2.4. Traffic Implications

Traffic characteristics (volume, speed limit, automobile vs truck volume) along a road section also can impact the evaluation of rockfall stability. The potential size of the rockfall is important. If the highway section is used by heavy trucks such as logging trucks, semitrailers or "B-Trains," significant wind as well as ground vibrations can be produced. The number or frequency of heavy traffic will be the dominant factor when considering this as a possible cause of smaller rockfalls.

The grade and topography of the highway also can be important. A semitrailer vehicle will use an engine brake on a long, steep downgrade. If a box cut exists on this stretch of highway, the echo from the "jake" brake can reverberate significantly in the area.

Speed limit, natural traffic flow speed, and visibility influence the potential stopping distance for a vehicle whose driver sees an obstruction on the highway. Curves reduce sight visibility. Therefore, on highways with high volume and speed, near horizontal curves and humped vertical curves, extra attention should be given to rockfall control (figures 4-3 and 4-4).

The slope geometry angle, roughness, height, and gully location will influence the potential rockfall trajectory. Potential rock catchment is determined by the ditch capacity to catch and hold rock (width, depth, and shoulder-side slope). Therefore, slope and ditch sections should be measured and plotted. Figures 4-3 and 4-4 show very narrow catch ditches, whereas figure 4-5 shows a wide catch ditch.

At numerous locations, the outer road shoulder will be above steep slopes, gullies, or water. Should the vehicle leave the highway, the potential of serious injury, death, and damage is increased. Thus, topographic details of the outer slopes should always be obtained.

4.2.5. Air Photo Interpretation

Air photo interpretation can be very effective for recognizing potential hazards of any type. This method provides a three-dimensional view of the terrain and associated topography. The amount of information that can be gathered from air photos is primarily dependant upon the reviewer's qualifications and experience, characteristics of the air photographs, such as scale, quality and type of air photos, and whether they are oblique or vertical, black and white, or color or infrared.

The principles of air photo interpretation involve identification of specific landforms from a unique signature produced on the surface. These unique patterns are comprised of several characteristics, including topographic expression, structural geology, rock type, drainage, erosion, soil tones, and vegetation.

Although the causes of rockfalls are different, the resulting appearances after movement are usually similar. Air photo interpretation can assist in the identification of fresh irregular scars that mark the fall source, the path, and possibly some indication of the frequency from the presence or absence of vegetation (figure 4-6). It also can assist in the identification of tree types that can disclose the severity or duration of a rockfall by the absence of long-lived trees, such as the conifer species, and the presence of much faster-growing trees, such as aspen.

Figure 4-1. Cliffs of hard, blocky rock above a talus slope that has been undercut. High rockfall momentum can develop *(Courtesy Colorado Department of Transportation).*

Figure 4-2. Sandstone cliffs above a talus and colluvial slope. Note the rockfalls at the toe *(Courtesy Colorado Department of Transportation).*

Figure 4-3. The sharp curve with a small catch ditch below. Large blocks on the rock face presents a dangerous traffic hazard should rockfall occur.

Figure 4-4. Visibility is severely restricted at this location. Note the vegetation at the crest of the slope that can lever rock onto the highway.

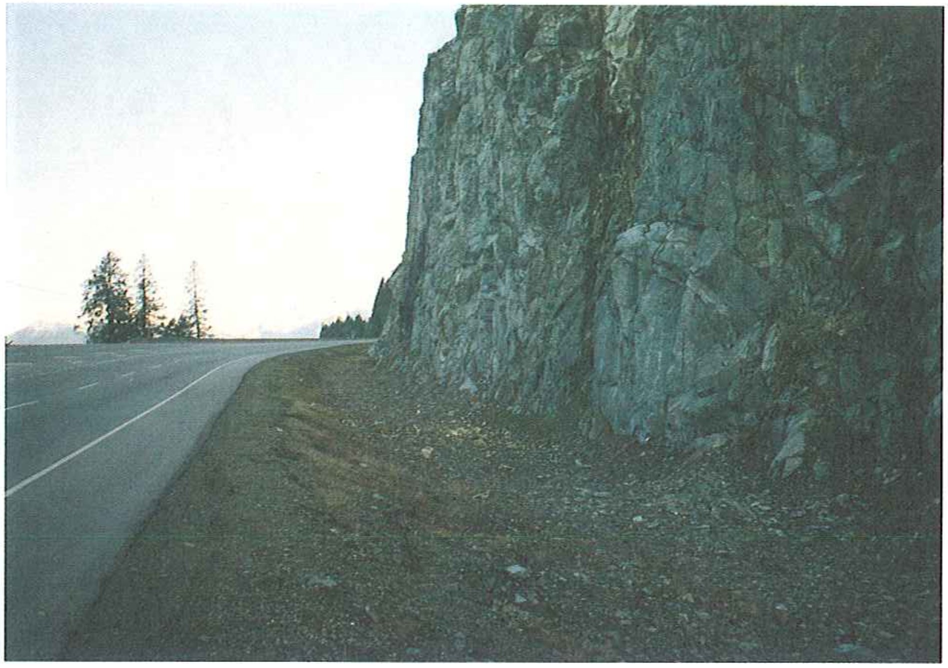


Figure 4-5. A wide catch ditch below a near-vertical rock slope. Most rock will be caught in the ditch.

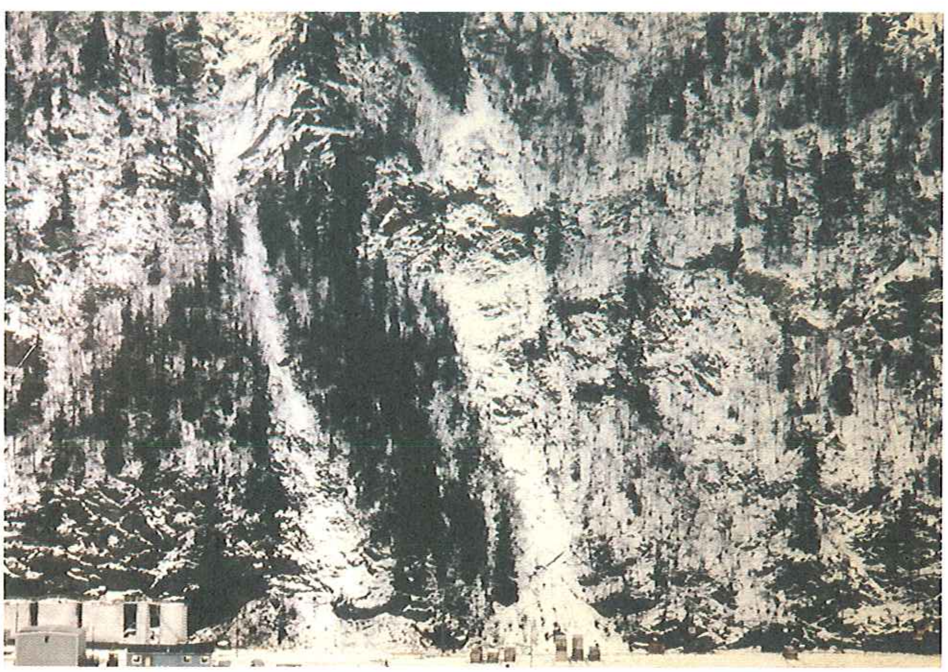


Figure 4-6. Vegetation denuded by ravelling rock. Note the talus toe slopes below each of the ravelling paths. Rock entrapment or catchment is recommended at the ditch level. Photo taken from an aircraft landing at the airport, Skagway, Alaska.

Air photos can help to determine the potential rockfall hazard at the crest of the slope. During field reconnaissance, observation of a slope face from highway level is common and the higher elevations and crest of the slope can not easily be inspected. Air photos from the proper scale and angle to the slope can provide the coverage of the slope crests to enable an experienced reviewer to identify areas that may require closer investigation for identification of loose blocks or areas of potential toppling or spalling. Air photos also can identify and locate danger trees on the slope and at the crest.

Oblique and ground photography can help identify more subtle features of the slope than can vertical air photos typically taken from aircraft. Ground and oblique stereo pairs can be taken using a 35mm hand-held camera. Stereo pairs are obtained by photographing the site from two points about 10 feet (3.5 meters) apart. Or, photographs from a helicopter or airplane can be used to measure distances and the structural dip and orientation of exposed joint surfaces by using ortho-photography techniques.

Infrared photography is useful for two reasons. The first is the relative ease in identifying water seepage from the bluish tinge seen on the photographs. The second is the ability of this type of photograph to enhance hidden fissures and cracks and faulted areas. Since fissures generally act as water "sinks," dull red colors represent inhibited growth, while springs and shear zones that hold water and feature more abundant growth are represented by a more brilliant red.

4.3. SURFACE GEOTECHNICAL INVESTIGATION

As discussed in chapter 2, the importance of recognizing the influence of structural geology, groundwater, and weathering is crucial in the prevention and evaluation of a rockfall area. These factors are primarily identified from geologic reconnaissance of the existing slope. The objective of any surface investigation should be to define as well as possible the structural geologic geometry of the site in three dimensions.

4.3.1. Visual Examination

A visual examination of the overall slope and crest is mandatory in any stability investigation. This should be performed prior to any detailed geologic investigations but after review of material gathered during the literature search and air photographs. Because most initial investigations occur at grade, the slope crest can be neglected. The crest of the slope should always be investigated to determine the rock mass characteristics and the potential for rockfalls. If the site is very steep or high, inspection by helicopter can be useful (figure 4-7). The visual assessment also can determine other areas where investigations should be concentrated, as well as observations of seepage areas, ice formations, and pothole damage in the asphalt surface (figure 4-8).

In some instances, observation and geologic measurement of the slope cannot adequately be made from grade. In these instances, give consideration to using hydraulic manlifts, bucket trucks, or mountaineering techniques to rappel the slope, (figure 4-9) measure structural geology, and locate potentially unstable rocks to be stabilized or romoved by scaling.

4.3.2. Rock Type and Condition

Outcrops observed on the slope can provide information on the condition and type of rock. As discussed in chapter 2, knowledge of the different lithologies and the degrees to which weathering and alteration have occurred will provide a basis for estimating how the rock mass will behave. Each lithologic type will have its own unique properties, such as macro and micro structural discontinuities, mineral assemblage, weathering, and texture.

For rock type identification, it is important to review geologic reports of the area to get an idea of what lithologies are to be encountered. If no geologic information is available, then it may be advisable to confirm the rock type by obtaining a sample for thin section analysis by a geologist.

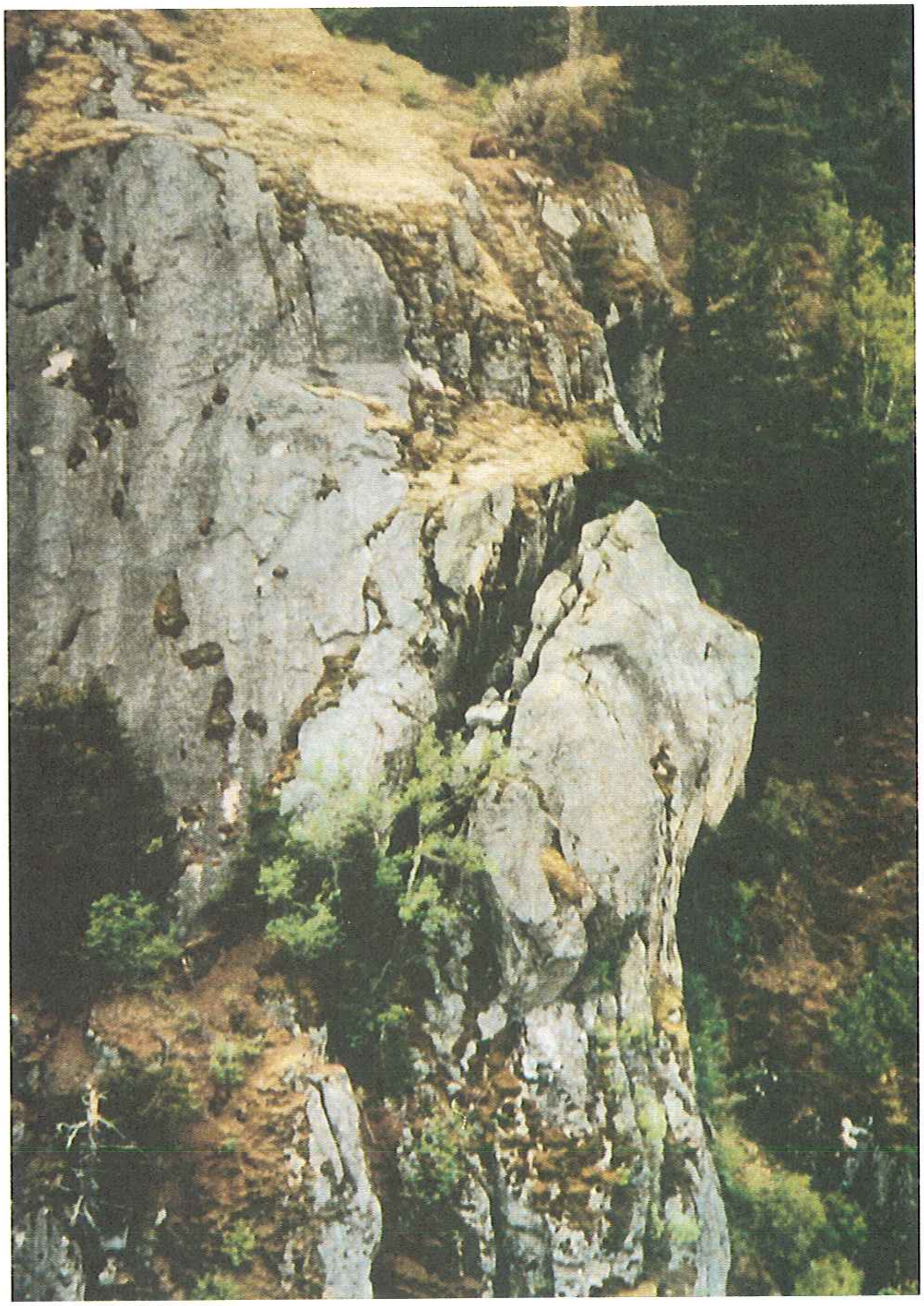


Figure 4-7. A major tension crack has opened on the face of a granite cliff about 150 feet (45.8 meters) above traffic. A serious toppling potential exists. This crack cannot be observed from grade level or safely from the crest of the slope. A helicopter inspection revealed this dangerous condition.

Figure 4-8. Fingerprints of rockfall on the asphalt pavement. Such evidence indicates past rockfall or rock removal due to scaling.

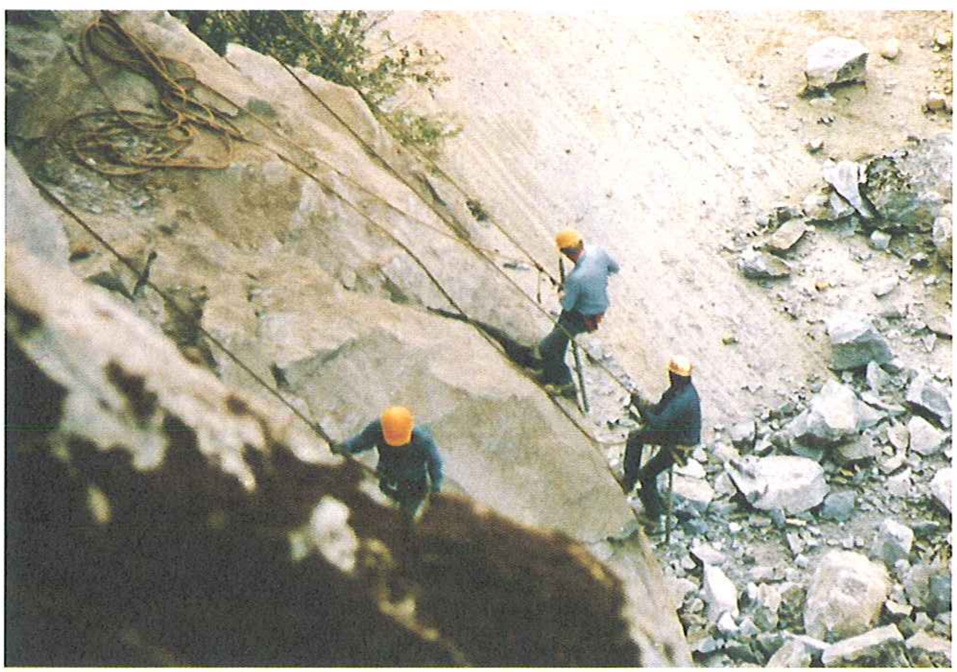


Figure 4-9. Rock inspection by rope (rappelling) and scaling loose rock beside a planar slide zone. This work must be performed by experienced technical rock climbers *(Courtesy Washington Department of Transportation).*

The intact rock strength can significantly impact the shear strength of the rock, especially if there is no infilling material. Weathering also alters the strength characteristics of the rock. During the field investigation, it is important that the weathering characteristics of the rock type be identified. A useful field technique is to use Deere and Patton's (1979) hardness chart and updated by Robertson (1987) and shown on table 2-1. This chart incorporates a correlation between rock hardness, typical field identification, and unconfined compressive strength.

4.3.3. Structural Geology

The structural characteristics of the rock mass require the most detailed investigation. With the potential for instability being much greater in areas of extensive faulting and jointing, characteristics of all discontinuity types, such as faults, joints, and bedding, are required to properly model the rock mass. Identifying these areas is important during preliminary design and site or route selection.

Geologic mapping is the most commonly used tool for determining the three-dimensional structural characteristics of a slope. Mapping methods such as detailed line mapping, fracture set mapping, or cell mapping have been used for identification of specific features. All three methods are adequate, but it is recommended that the geological engineer pick a particular mapping method that most appropriately suits the slope situation.

One of the most important tools in determining patterns in the structural geology of a site is to use stereographic analysis (Goodman, 1976). This type of analysis utilizes the projection of poles of structural planes on stereonets to produce a three-dimensional representation of the field data. This type of data representation can also be used to determine various failure types (figure 4-10).

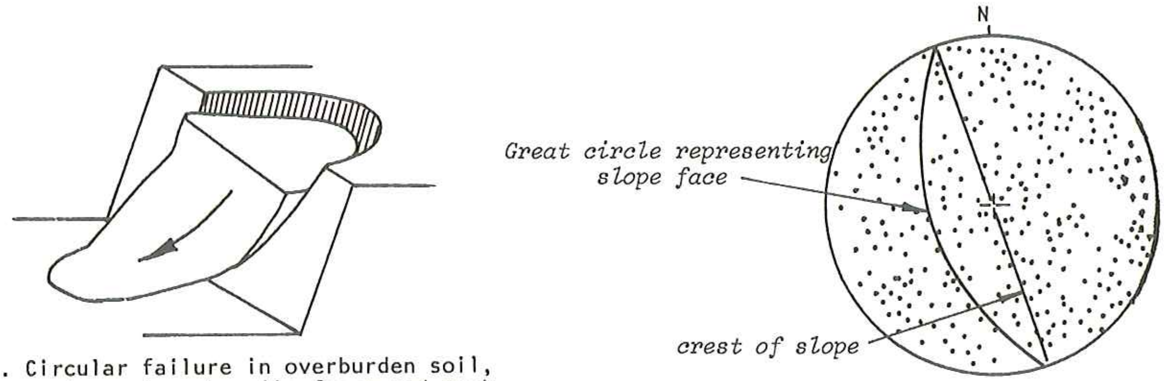
Faulting can be the most crucial element for slope stability since it can influence a very large area of the rock mass through weathering and sympathetic jointing. The dip direction, angle of dip, infilling characteristics, continuity, roughness, and wall rock condition should be recorded during the field investigation.

Joint sets also require the recording of dip, orientation, continuity, spacing, infilling characteristics, and aperture. A Brunton or Clar compass is recommended for field mapping. Statistical representations of the joint orientations can assist greatly in determining the influence of these systems on the slope. Closely spaced joints are typically found in areas of rockfall hazard. Figure 4-11 is a typical discontinuity mapping form.

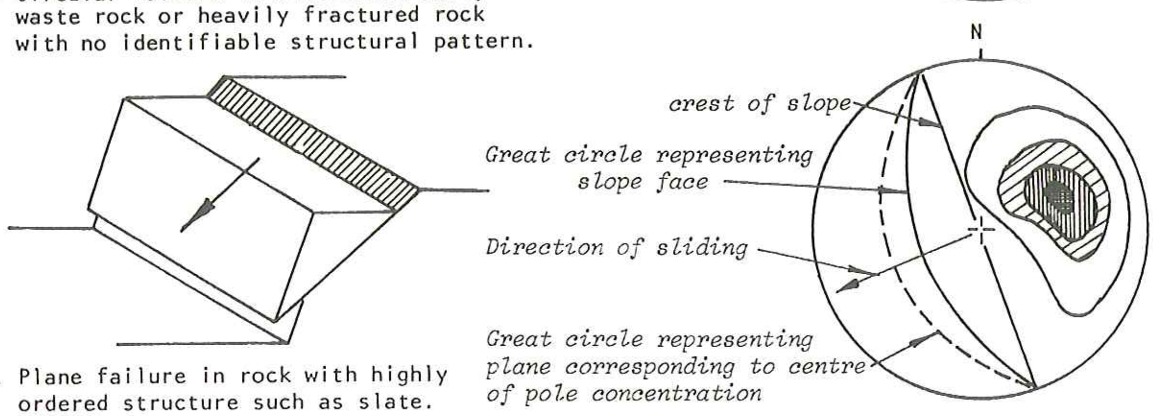
Bedding and foliation in sedimentary and metamorphic type rocks will have a dominant impact on slope stability. Sedimentary deposits frequently display bedding, which produces jointing and preferential seepage paths for water to travel that can influence stability of a slope. Tectonic stresses and metamorphism can alter the orientation of structural discontinuities and careful study of the slope to establish areas where mapping should be concentrated is recommended.

The field mapping should locate and plot tension cracks that can be observed (figure 4-12). The size of any potential fall associated with cracking should be estimated.

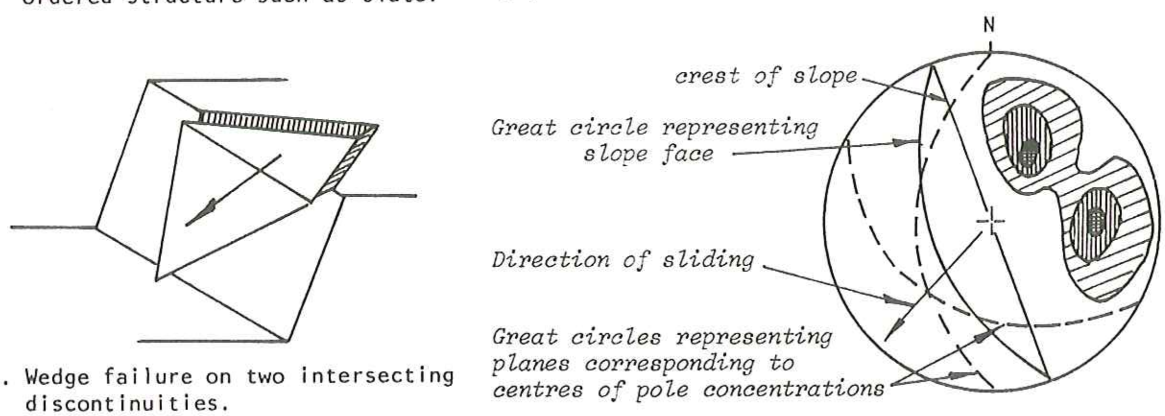
Document evidence of any light-colored surfaces that would indicate recent rockfall (figure 4-13). It generally takes a fresh rock face about 10 years to weather to the color of long-term exposures. This provides a qualitative measure of the time of past rockfalls.



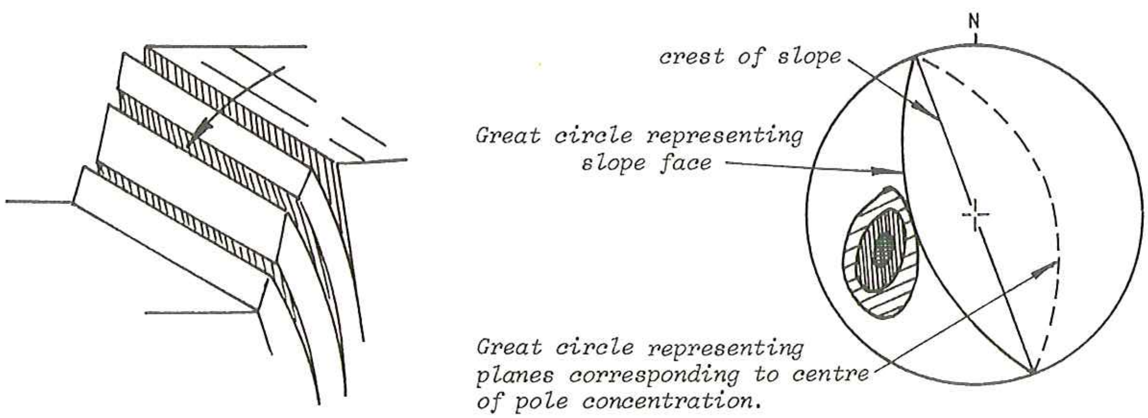
a. Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern.



b. Plane failure in rock with highly ordered structure such as slate.



c. Wedge failure on two intersecting discontinuities.



d. Toppling failure in hard rock which can form columnar structure separated by steeply dipping discontinuities.

Figure 4-10. Main types of slope failure and stereoplots of structural conditions likely to give rise to these failures (Hoek and Bray, 1981).

DISCONTINUITY DATA

* 0 2 Card Type										Identification				
No.	Discontinuity				Size	Spg.	Flng.		Water	Waves	Line		Lthlgy.	Hrdn.
	Distance	Type	Strike	Dip			1	2		I L A	Type	Pitch		

Figure 4-11. Typical discontinuity mapping form (Canmet, 1977).

Figure 4-12. Tension crack opened up about 12 inches (305mm). This is an obvious sign of serious instability. The movement is a combination of sliding on shale layers and toppling. Note the bridge below. The slope is so badly broken that excavation back to a stable slope is being recommended. Movement monitoring has been installed to monitor stability until a contract can be called and completed.

Figure 4-13. Light-colored faces from which rockfalls have occurred. The fresh scar on the left is recent (within one year). The scar on the right is less fresh and the fall likely occurred 3 to 4 years ago.

Where a new highway location is planned and few or no rock outcrops exist, structural geologic rock data must be obtained from a borehole drilling and coring program. Table considerable care to obtain as close to 100 percent undisturbed core recovery as possible. The core that is not obtained will likely be that which will have the greatest impact on stability. To increase core recovery, hydraulic drills and the wireline system with triple tube core barrels should be used. Where drilling is contracted, the experience of the drill foreman should be an important contract criterion. For optimum core recovery, the drill contract should specify payment by the hour with an incentive for maximum core recovery and minimum disturbance.

To evaluate structural geology, the core must be oriented. There are numerous procedures available, for example, Borehole camera, TV or periscope, Christiansen core barrel, and Craelius core orientor. The Call clay imprint method is recommended as the most cost effective and easiest to use (figure 4-14) (Call, Savely and Pakalnis, 1982).

Presentation of the data should include location, depth, dip and dip direction, spacing of discontinuities, width of discontinuity, gouge or infilling, surface roughness, rock quality, designation (R.Q.D.), and rock description. A typical structural core log data sheet is shown in figure 4-15.

4.3.4. Discontinuity Roughness

Shear strength of the rock mass is greatly affected by the surface roughness of a discontinuity; therefore, this characteristic becomes an important parameter for design. The peak strength of a discontinuity is mobilized when the projections along the joint surface are sheared. Beyond this point, the strength decreases rapidly and approaches the residual strength. The reduction in shear strength can be substantial.

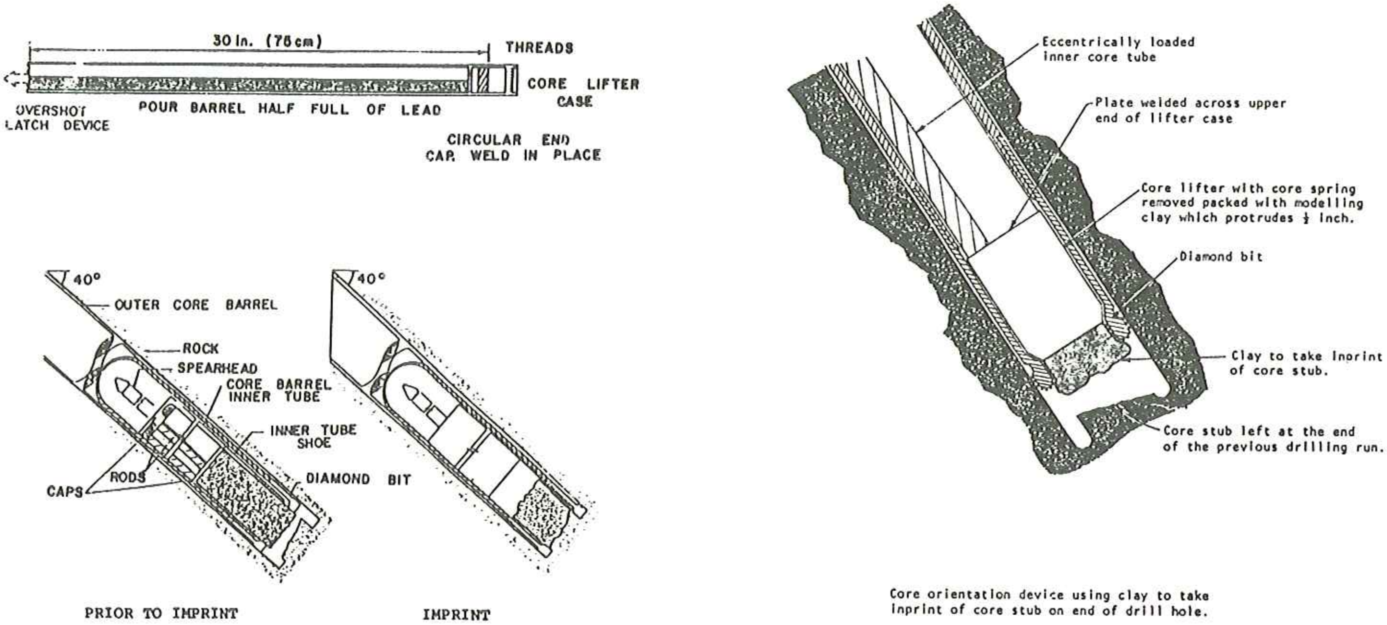


Figure 4-14. Call clay imprint method to orient drill core (Call, Savely, and Pakalnis, 1982).

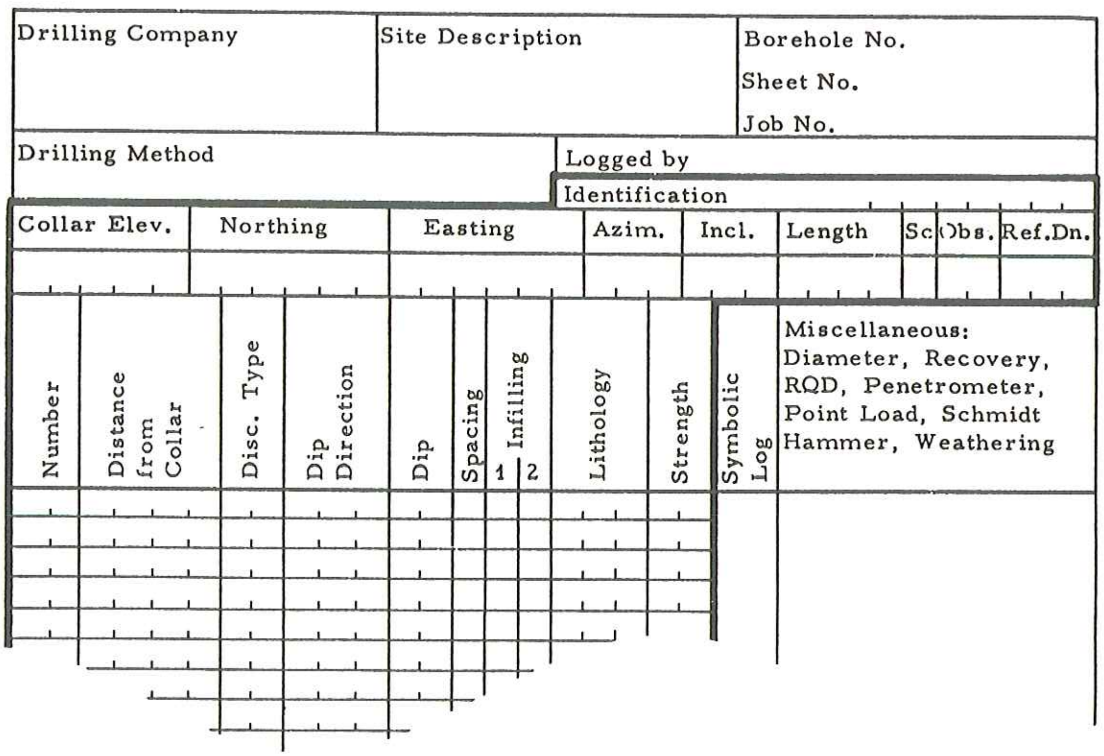


Figure 4-15. Typical rock structure core log (Canmet, 1977).

Patton (1966) found that to obtain a reasonable degree of agreement with observations in the field, the sum of the basic friction angle (ϕ) and the roughness angle ($\underline{i}$) of the asperities was required. Figure 4-16 defines what Patton established as first order projections and consists of the major undulations on the discontinuity or bedding planes. Higher values of $\underline{i}$ are obtained from measurement of the second order projections.

Barton (1973) of the Norwegian Geotechnical Institute (NGI) found that the shear strength of discontinuities is dependant not only upon the asperities of the discontinuity surface but also upon the normal stress applied on the plane. A higher normal stress would cause second order undulations along the plane to be sheared and an increased shear strength as compared to very low normal stress levels. Barton developed the Joint Roughness Coefficient (JRC), which relates shear strength and discontinuity to the roughness profiles shown in figure 4-17.

There are several methods of determining the second order surface roughness. A practical method is to hold the discontinuity in front of a projector, project the image on a wall, and draw the irregular shadow surface on a distant wall. The profiles can be traced for comparison to Barton's profiles (figure 4-17) or to enable the measurement of the roughness $\underline{i}$. Photographs of surface roughness are shown on figures 2-7 and 2-8.

4.3.5. Evidence of Past Instability

Evidence of past instability can alert the investigator to areas of future risk. Potholes and damage to the surface pavement of a highway or roadway define areas of a rockfall hazard. Other clues include the accumulation of material in roadside ditches or benches, the absence of vegetation similar to that of an avalanche track, and a fresh rock surface exposed on a face. Evidence of past instability is best determined by someone familiar with the geology and conditions in the particular area.

4.4. OTHER INVESTIGATIONS

Other information may be useful prior to design of remedial measures.

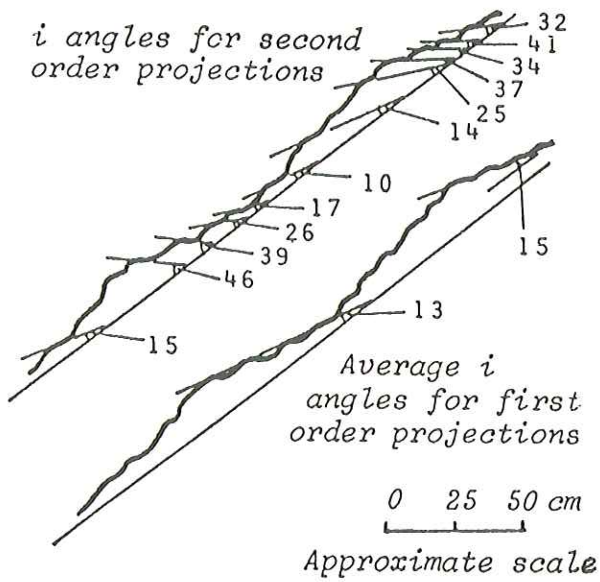
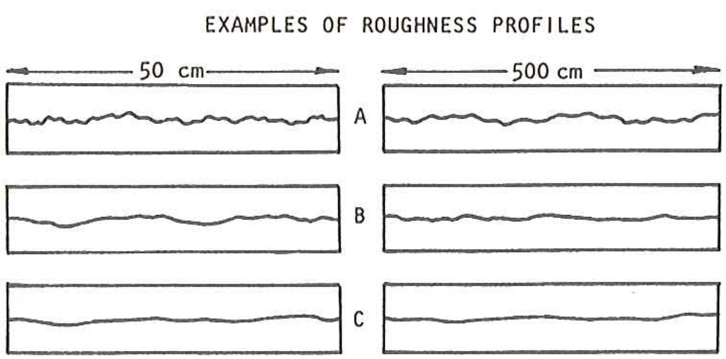


Figure 4-16. Patton's measurement of *i* angles for first and second order projections on rough rock surfaces. (Patton 1966)



A. Rough undulating - tension joints, rough sheeting, rough bedding. JRC = 20

B. Smooth undulating - smooth sheeting, non-planar foliation, undulating bedding. JRC = 10

C. Smooth nearly planar - planar shear joints, planar foliation, planar bedding. JRC = 5

Figure 4-17. Barton's definition of Joint Roughness Coefficient JRC (Barton 1973).

4.4.1. Water Chemistry

Water chemistry can have an important influence on type and cost of support in a particular area. Rock bolts and dowels can corrode where groundwater is acidic and cause maintenance and safety problems in the long term. Rock bolts that are pretensioned and not properly protected tend to corrode at a faster rate than the grouted dowel-type support. The additional cost of protecting support from corrosion is well worth the effort when considering long-term maintenance and stability. Adverse water chemistry, such as sulphates, also can cause cement grout to deteriorate.

4.4.2. Field Trials

Because it is difficult to predict the path of falling rock as it travels down the slope, field trials to determine the response can be extremely useful when determining measures to protect motorists. Examples of this type of investigation were performed by Ritchie (1963) and Piteau and Peckover (1978). Ritchie developed an empirical model to determine if rockfalls would bounce or roll down a slope and he used this to develop ditch design criteria (chapter 6).

These criteria were not developed for presplit slopes, which would be conservative and expensive.

Piteau and Peckover developed a computer program to simulate several hundred rockfalls on various slope angles. Rockfalls are introduced at various locations along the slope and probability factors are assigned to recognize areas that are more or less likely to be a source. The program currently recommended, Rockfall Simulation Program (1991), was developed by the Colorado Department of Transportation.

Once a field trial has been decided upon, safety of those observing the experimental program becomes the most critical issue. Video recording of the trials with a high-speed camera is the best technique to record the rock response. A measurement of distance travelled and location of impact from the slope will also provide the necessary information to determine ditch widths and depth. The use of field trials and correlation with computer design is presented in chapter 6.

4.4.3. Rockfall Check List

A check-list format for field inspection staff is very useful to ensure a field investigation is thorough. Several States have developed such lists. Since rockfall conditions in various States will differ, the check lists should recognize these various conditions.

As an example the Rock Fall Field Check List developed by the Alaska Dept. of Transportation is included on the next pages.

Note that this check list does not include circular failure. The majority of Alaska has been heavily glaciated so the depth of weathering is very shallow, a condition not conducive to circular failure in rock. In areas in the United Sates where glaciation has not occurred, particularly in the South, circular rock fall failures in weathered rock is quite common.

Each State should develop a check list that reflects conditions in that State. Such a separate check is very useful when developing information to use the Hazard Rating Manual.

ALASKA DOT&PF ROCK FALL FIELD CHECK LIST

Highway: ________________ Milepost: __________ Project: ______________
Compiled by: ________________________________ Date: ______________

1.0 CAUSES OF ROCKFALL

1.1 STRUCTURAL GEOLOGY AND STABILITY

ROCK FAILURE IS GENERALLY CONTROLLED BY THE FOLLOWING DISCONTINUITIES: NUMBER SEPARATELY OR IN COMBINATION IN ORDER OF PRIORITY.

YES	NO	POSSIBLY	
____	____	____	JOINTS
____	____	____	BEDDING PLANES
____	____	____	FOLIATION
____	____	____	SHEAR ZONES
____	____	____	FAULTS
____	____	____	FRACTURES CAUSED BY BLAST DAMAGE

WHAT IS THE MAJOR ROCK TYPE? ______________________________

1.2 TYPES OF FAILURES

YES	NO	POSSIBLY	
____	____	____	PLANAR
____	____	____	WEDGE
____	____	____	BLOCK
____	____	____	TOPPLING
____	____	____	BUCKLING
____	____	____	KEY BLOCK
____	____	____	RAVELLING
____	____	____	WEATHERING

1.2.1 PLANAR FAILURE

YES	NO	POSSIBLY	
____	____	____	PLANE OF DISCONTINUITY DIPS OUT OF SLOPE
____	____	____	DISCONTINUITY IS UNDERCUT BY EXCAVATION
____	____	____	DIP ANGLE EXCEEDS ANGLE OF FRICTION ALONG DISCONTINUITY

____	____	____	STRIKE OF DISCONTINUITY LESS THAN 20° FROM STRIKE OF EXCAVATION FACE
____	____	____	TENSION CRACK DEVELOPED AT THE TOP OF SLOPE

1.2.2 WEDGE FAILURE

YES	NO	POSSIBLY	
____	____	____	TWO OR MORE DISCONTINUITIES INTERSECT AND DIP OUT OF SLOPE
____	____	____	TOE OF WEDGE DAYLIGHTS IN THE SLOPE
____	____	____	DIP ANGLE OF INTERSECTION EXCEEDS AVERAGE FRICTION ANGLE ALONG DISCONTINUITIES
____	____	____	TRANSIENT WATER PRESSURES DEVELOP IN THE DISCONTINUITIES
____	____	____	MAJORITY OF WEDGE FAILURES ARE SMALL (>4')

1.2.3 BLOCK FAILURE

YES	NO	POSSIBLY	
____	____	____	WEAK HORIZONTAL LAYERS OR BEDDING EXIST
____	____	____	WEAK LAYER GENERALLY CLAYEY TYPE ROCK (OFTEN BENTONITIC) OR WEAK SCHIST
____	____	____	TENSION CRACK DEVELOPS AT BACK OF BLOCK -- USUALLY DUE TO STRESS RELIEF
____	____	____	WATER PRESSURE IN TENSION CRACK CAUSES FAILURE
____	____	____	DEPENDING ON FRICTION ANGLE, BLOCK CAN MOVE ON SLIGHTLY UPHILL PLANE

1.2.4 TOPPLING FAILURE

YES	NO	POSSIBLY	
____	____	____	STEEPLY INCLINED DISCONTINUITIES
____	____	____	GENERALLY STEEP CUT SLOPE FACE
____	____	____	WATER PRESSURE DEVELOPS IN DISCONTINUITIES
____	____	____	FLEXURAL TOPPLING -- GENERALLY BASE OF SLOPE
____	____	____	BLOCK TOPPLING -- SEPARATE BLOCKS
____	____	____	RAVELLING

1.2.5 RAVELLING FAILURE

GENERALLY ONE TO SEVERAL ROCKS CAUSED BY:

YES	NO	POSSIBLY	
____	____	____	ICE JACKING
____	____	____	TREE ROOTS PUSHING
____	____	____	VIBRATION (TRAINS, EARTHQUAKE)
____	____	____	WEATHERING
____	____	____	ANIMALS

1.2.6 DIFFERENTIAL WEATHERING AND EROSION

YES	NO	POSSIBLY	
____	____	____	WEAK OR WEATHERED LAYER UNDERLYING MORE COMPETENT ROCK (COMMON WHERE MULTI-LAYER VOLCANIC AND INTERBEDDED SHALES AND SANDSTONES OCCUR).
____	____	____	EROSION OF SILT/SAND SOIL MATRIX RELEASES LARGER SIZE COBBLES AND BOULDERS.

1.3 WEATHERING DUE TO:

YES	NO	POSSIBLY	
____	____	____	FREEZE THAW CYCLES
____	____	____	WET DRY CYCLES
____	____	____	TEMPERATURE CYCLES
____	____	____	OXIDATION
____	____	____	ACID REACTION

1.4 TREE ROOTS IN DISCONTINUITIES

YES	NO	POSSIBLY	
____	____	____	WEDGE BLOCKS APART
____	____	____	WINDS CREATE HIGH LEVERAGE FORCES

1.5 VIBRATION

YES	NO	POSSIBLY	
____	____	____	EARTHQUAKE
____	____	____	CONSTRUCTION EQUIPMENT
____	____	____	TRAFFIC

1.6 STRESS RELIEF

YES	NO	POSSIBLY	
____	____	____	EROSION
____	____	____	EXCAVATION
____	____	____	TIME FACTOR IN CLAYEY ROCKS
____	____	____	DIFFERENTIAL STRESS RELIEF AT CHANGE IN ROCK TYPES

1.7 INFLUENCE OF GROUNDWATER ON ROCK SLOPE STABILITY

GROUNDWATER INFLUENCE:

YES	NO	POSSIBLY	
____	____	____	REDUCES FRICTIONAL SHEAR STRENGTH $S = N \tan Ø$ BECOMES $S = (N-U) \tan Ø$
____	____	____	REDUCES COHESIVE SHEAR STRENGTH IN WEAK TO MEDIUM STRENGTH ROCKS, FAULT GOUGE AND INFILL MATERIALS
____	____	____	SEEPAGE FORCES
____	____	____	WATER PRESSURE IN TENSION CRACKS
____	____	____	FREEZE THAW -- ICE JACKING

1.8 INFLUENCE OF BLASTING ON STABILITY

USE OF UNCONTROLLED BLASTING DAMAGES ROCK -- ROCK DAMAGE YIELDS FLATTER, UNSTABLE SLOPES WITH MUCH MORE LONG-TERM ROCKFALL AND MAINTENANCE REQUIRED

YES	NO	POSSIBLY	
____	____	____	IMPROPER BLASTING FRACTURES AND LOOSENS ROCK MAKING IT MORE SENSTIVE TO THE PREVIOUSLY DESCRIBED CAUSES OF ROCKFALL
____	____	____	OPENS EXISTING DISCONTINUITIES -- REDUCES EFFECTIVE ANGLE OF INCIDENCE (ROUGHNESS)
____	____	____	BREAKS INTACT ROCK (ROCK BRIDGES)
____	____	____	CREATES RAVELLED BENCH CREST
____	____	____	MUST REDUCE SEISMIC BLAST ENERGY NEAR FINAL FACE

2.0 SITE INVESTIGATION

YES	NO	POSSIBLY	
____	____	____	VISUAL EXAMINATION OF SLOPE
____	____	____	MEASURE STRUCTURAL ORIENTATION OF DISCONTINUITIES
____	____	____	TALK WITH MAINTENANCE
____	____	____	ROCKFALL HISTORY
____	____	____	FREQUENCY OF OCCURRENCE
____	____	____	SIZE OF ROCKFALL
____	____	____	QUANTITY PER EVENT
____	____	____	OBSERVE AREAS OF SEEPAGE OR ICE BUILD-UP ON SLOPE FACE
____	____	____	OBTAIN WATER CHEMISTRY SAMPLES TO DETERMINE LEVEL OF CORROSION PROTECTION REQUIRED FOR ROCK DOWELS OR ROCK BOLTS (IF LIKELY TO BE USED FOR STABILIZATION)
____	____	____	USE OF AIR PHOTOS AND OBLIQUE PHOTOS
____	____	____	FIELD TRIALS -- ROLL ROCKS DOWN SLOPE
____	____	____	USE OF MOUNTAINEERING TECHNIQUES FOR CLOSE-UP EXAMINATION OF ROCKSLOPE
____	____	____	ENGINEERING GEOLOGIST RAPPELS DOWNSLOPE
____	____	____	PLANAR FEATURES CONTROLLING STABILITY EXAMINED AND MEASURED DIRECTLY
____	____	____	CHANGES IN DISCONTINUITY SPACING, PRESENCE OF WATER, OR ATTITUDE CAN BE MEASURED DIRECTLY
____	____	____	CAN ACCURATELY DETERMINE CRITICAL CONTROL POINTS ON SLOPE BY SURVEY CREW AT ROAD LEVEL SHOOTING TARGET HELD BY THE MOUNTAINEERING CREW

3.0 MONITORING

YES	NO	POSSIBLY	
____	____	____	VISUAL
____	____	____	COUNT NUMBER OF ROCKS THAT FALL OVER TIME
____	____	____	SLOPE MOVEMENTS
____	____	____	TRIPODS, PULLEYS, AND CABLES
____	____	____	EDM/THEODOLITE

____	____	____	HUBS AND MEASURING BAR
____	____	____	TILT PLATES
____	____	____	EXTENSOMETERS
____	____	____	WARNING FENCES

4.0 ANALYSIS

YES	NO	POSSIBLY	
____	____	____	ENERGY ANALYSIS
____	____	____	ROCKFALL TRAJECTORY ANALYSIS
____	____	____	ROCKFALL COMPUTER SIMULATION
____	____	____	FIELD TESTING -- ROLLING ROCKS DOWN SLOPE
____	____	____	DESIGN OF MITIGATION METHODS

5.0 ROCKFALL MITIGATION METHODS (STABILIZATION, PROTECTION, WARNING)

5.1 STABILIZATION METHODS

YES	NO	POSSIBLY	
____	____	____	EXCAVATION
____	____	____	SCALING
____	____	____	REMOVAL BY BLASTING
____	____	____	TRIM BLASTING
____	____	____	CRACK BLASTING
____	____	____	BOULDER POPPING
____	____	____	MUD CAPPING
____	____	____	FLATTEN SLOPE
____	____	____	DESIGN TO GEOLOGY
____	____	____	REDUCE BLASTING DAMAGE TO SLOPE FACE (FOR NEW ROCK CUTS OR RE-EXCAVATION OF EXISTING FACE)
____	____	____	USE CONTROLLED BLASTING (PRESPLIT OR PRESHEAR)
____	____	____	CLOSER SPACED LINE HOLES
____	____	____	USE OF BUFFER LINES
____	____	____	USE OF DELAYS
____	____	____	BLAST TO FREE FACE
____	____	____	WHEN CAN ALLOW ANGLE DRILLING AT FACE

___	___	___	DO NOT ALLOW USE OF "LIFTERS" OR UNCONTROLLED BLASTING
___	___	___	NON-BLASTING METHODS
___	___	___	CHEMICAL EXPANDERS
___	___	___	HYDRAULIC SPLITTERS
___	___	___	DRAINAGE
___	___	___	SURFACE DRAINAGE BY DIVERSION DITCH ABOVE CUT
___	___	___	SUBSURFACE DRAINAGE BY HORIZONTAL DRAIN HOLES
___	___	___	SUPPORT SYSTEMS
___	___	___	SHOTCRETE
___	___	___	STEEL FIBER REINFORCED
___	___	___	POLYPROPYLENE FIBER REINFORCED
___	___	___	WITH WELDED WIRE REINFORCEMENT
___	___	___	WITH SILICA FUME
___	___	___	ANCHORING
___	___	___	DOWELS
___	___	___	ROCKBOLTS AND ANCHORS (PASSIVE VS TENSIONED)
___	___	___	CORROSION PROTECTION
___	___	___	CABLE LASHING
___	___	___	BOLTED WIRE MESH
___	___	___	CONCRETE BUTTRESSES
___	___	___	RETAINING WALLS

5.2 PROTECTION METHODS

YES	NO	POSSIBLY	
___	___	___	SLOPE TREATMENTS
___	___	___	INTERMEDIATE SLOPE BENCHES (DO NOT USE -- CREATES "LAUNCHING RAMPS")
___	___	___	CATCH BERMS
___	___	___	DIVERSION BERMS
___	___	___	DRAPED WIRE MESH
___	___	___	DITCH TREATMENTS
___	___	___	CATCHMENT DITCH
___	___	___	DEEPER DITCH
___	___	___	WIDENING AT GRADE

____	____	____	ROCK PROTECTION FENCES
____	____	____	STANDARD ROCK PROTECTION FENCES (WASHINGTON STATE DOT DESIGNS)
____	____	____	ROCK FENCE WITH DRAPED MESH (OREGON STATE DOT DESIGN)
____	____	____	COLORADO STATE DOT FLEX FENCE
____	____	____	HEAVY DUTY ROCKNETS (BRUGG AND EI)
____	____	____	OTHERS?
____	____	____	ROCKFALL BARRIERS AND WALLS
____	____	____	METAL GUARDRAIL
____	____	____	CONCRETE JERSEY BARRIERS WITH AND WITHOUT FENCE
____	____	____	STEEL H-BEAM W/TIMBER LAGGING WALLS
____	____	____	GABIONS
____	____	____	MSE WALLS
____	____	____	RELOCATE ROADWAY
____	____	____	ROCK SHEDS
____	____	____	TUNNEL

5.3 WARNING METHODS

YES	NO	POSSIBLY	
____	____	____	SIGNS
____	____	____	ROAD PATROLS
____	____	____	ELECTRIC FENCES AND WIRE
____	____	____	MONITORING

The above rock fall field check list is intended to be used by the engineering geologist or geotechnical engineer investigating the rock fall case. It should be filled out in the field as soon after the reported rock fall as possible. A narrative should also be written.

CHAPTER 5

MONITORING

5.1. VISUAL INSPECTION

The most important first step in any monitoring program is a visual inspection. This inspection normally is first performed from the highway by a specialist rock mechanics, geotechnical or geological engineer, or engineering geologist. The engineer will take special note of the lithology, evidence of movement and structural geology and evaluate the potential for various types of rockfall. In some instances, the potential and danger is very obvious. In others, a detailed structural geologic mapping program may be considered necessary.

The engineer will look for evidence of actual instability, such as tension or shear cracks, loose rockfall in the ditches, or indentations in the road surface made by falling rock. Evidence and location of seepage should be noted. In winter, icicles and ice glaciers on the rock face should be recorded. Any danger trees-trees with roots extending into discontinuities on the slope or within about 6 feet (11 meters) of the crest-should be noted for removal.

An evaluation should be made of the existing potential instability and danger. The first formal inspection of rock stability in North America was performed for Canadian Pacific Railway in 1974 (Brawner, 1975). A check list, such as that shown at the end of chapter 4, should be completed and photographs of the more serious areas should be taken.

This information will form a data base to assist with the overall evaluation and rating system, as described in the FHWA Rockfall Hazard Rating Manual (1993). There may be some locations where stability is considered to be so critical that stabilization must be implemented as soon as possible.

Where evidence of potential instability exists well up on the slope, it is desirable to climb to the location and make a close-up inspection. In instances where the slopes are high or the faces are difficult to access, inspection by helicopter or geotechnical rock climbers is recommended. A very experienced pilot and maneuverable helicopter are essential since maneuvers are usually required near the rock face with some wind.

It is desirable that department of transportation staff, such as maintenance supervisors who frequently travel the highways be given a training program in elementary structural geology, rock mechanics, and rockfall mitigation to identify potentially unstable conditions. A color-slide presentation of typical examples is most effective. Experience has proven that these staff take a very keen interest in field observation and identification of potentially unstable conditions.

5.2. SLOPE MOVEMENTS

5.2.1. Direct Crack Monitoring

Where cracks have opened up on the slope and can be observed from the highway, a simple procedure to determine if movement is occurring is to drive a pointed survey stake or stakes into the crack and observe over time if the stake falls out (figure 5-1). The stake should be tied to a short wire attached to a long nail that is driven into a nearby joint so the dislodged stake remains near the crack when the movement causes it to fall out. For better observation, the stakes should be painted a color that is easily observed. As the maintenance staff travels the highway, they should routinely observe the stake. If it has dislodged, they must immediately advise the senior State geotechnical engineer or engineering geologist.

Numerous other techniques are available to monitor open cracks on the slope or beyond the crest. These include measurements between grouted bars on either side of the crack, grouted movement plates or use of wire extensometers (figures 5-2, 5-3, and 5-4).

5.2.2. Remote Monitoring

In remote areas, automatic movement measurement recorders with radio-transmission capability can be installed (figure 5-5). This system usually ties into a computer and printer (figure 5-6). The program includes acceleration criteria, which if exceeded, initiates a warning (Modular Mining Systems).

Figure 5-1. A wedge-type failure has developed with movement of about one inch (25.4mm). A black painted survey stake has been driven into the crack. The stake is tied to a wire. If the stake falls out, movement is indicated and the wedge should be removed or stabilized.

Figure 5-2. Rebars grouted into either side of a tension crack. Horizontal and vertical components are measured to determine the directional component of the movement.

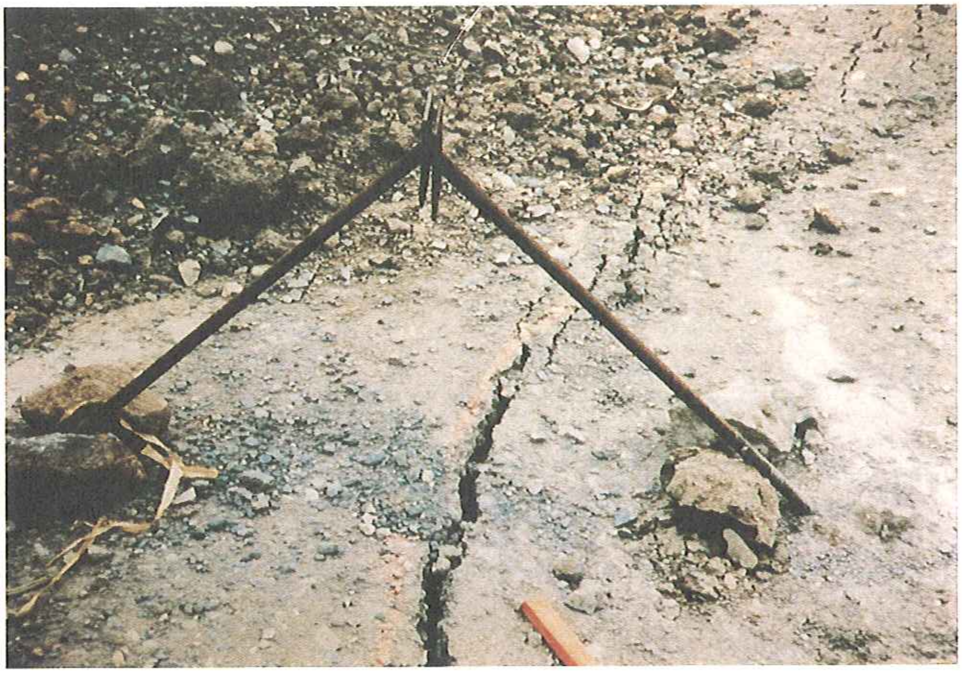


Figure 5-3. Grouted movement plates installed on either side of a tension crack. Provided the plates are exactly opposite one another to start, the horizontal and vertical movement components will assist in determining the type of movement. Periodic measurement will determine if the movement is accelerating and stability is reducing.

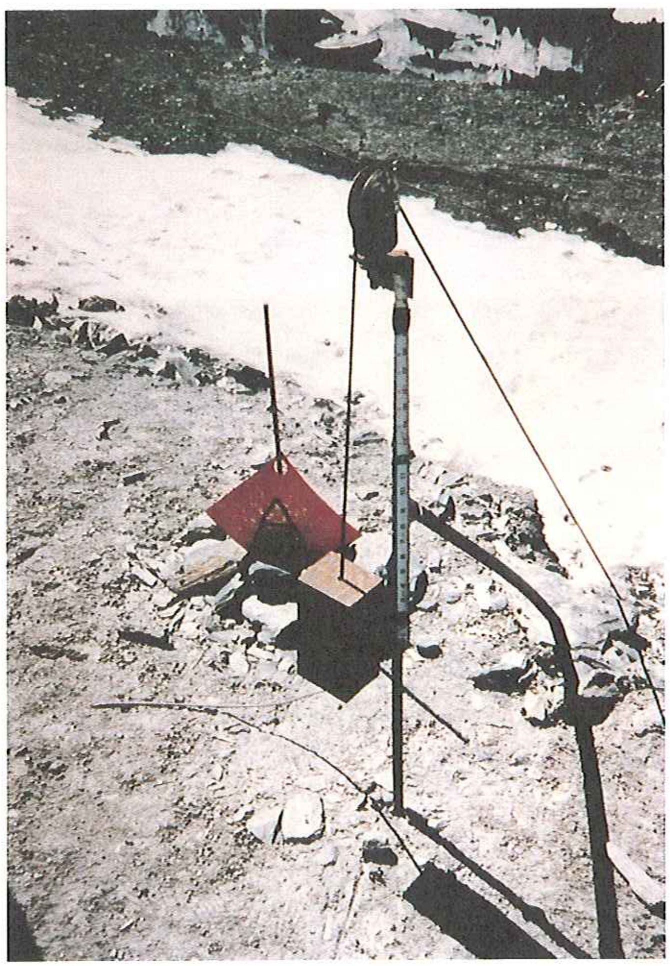


Figure 5-4. Wire line extensometer across a crack. Periodic measurement will indicate if the movement is increasing.

Figure 5-5. Automatic reading wire line extensometer with radio sending capability. The unit can be set to read at any time interval. The radioed data is picked up and plotted in a control room office (Modular Mining Systems).

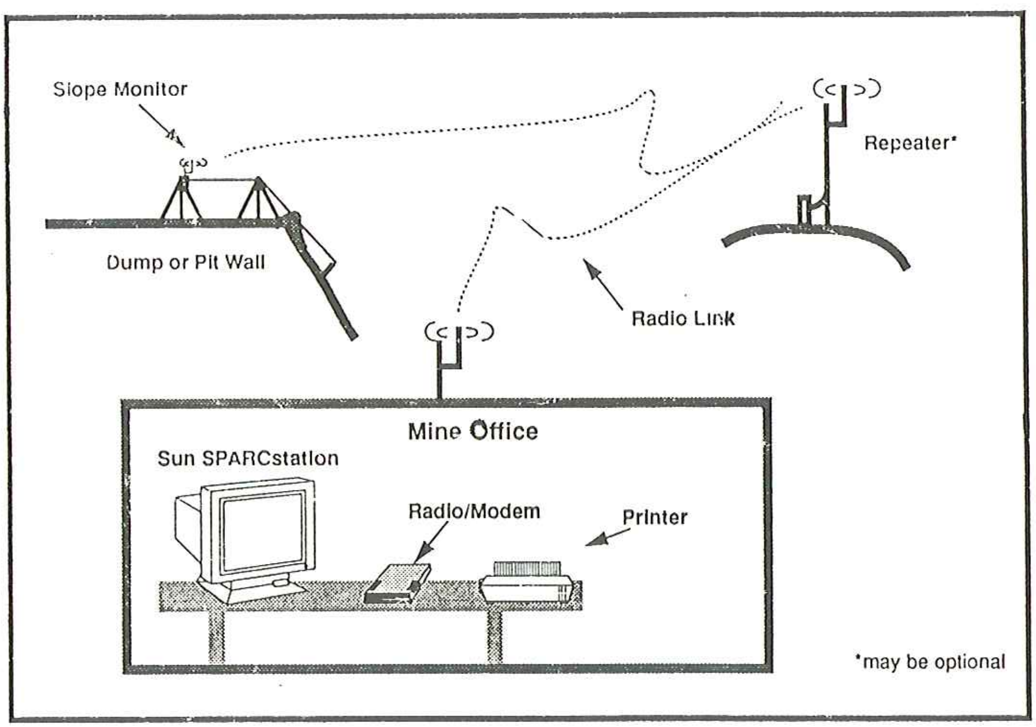


Figure 5-6. Schematic diagram of an automatic reading wire line extensometer with radio transmission to a computer printer facility (Modular Mining Systems).

Remote movement measurement using Electronic Distance Measurement (E.D.M.) technology will have application where relatively large rock masses appear to be potentially dangerous and exist well above the highway. This system has largely replaced triangulation survey procedures. The E.D.M. system combines reflecting mirrors installed on the potential moving area. The Electronic Distance Measurement unit is set up on a known stationary location. Distances are determined by measuring the phase difference between transmitted light beams using the laser principle. Many companies now manufacture these units.

For short distances, small mirrors or bicycle reflectors are usually adequate. For distances more than 200 to 300 feet (61 to 91.5 meters) to several miles, special mirrors with 5 to 6 faces at different angles should be used (figure 5-7). The multi-angled faces are necessary to obtain ongoing readings when the rock is moving.

E.D.M. units can be obtained with a theodolite component so angles as well as distances can be measured (figure 5-8). This capability is necessary when the potential rockfall is large or when the type of movement (chapter 2) is required. A very recent development is the self reading transmitting E.D.M. system. The E.D.M. is programmed to sight itself on respective mirrors and automatically take distance readings. These readings are transmitted from the E.D.M. to a radio pickup or telephone pickup where it is connected to a computer and printer at a control location. This location may be hundreds of miles away. The system can be programmed to take periodic readings over a 2 to 4 hour period and to flag acceleration of movement as a warning.

The accuracy of the system will vary depending on the system used. For small volumes, an accuracy of at least 0.25 inches (6.35mm) is desirable. For large rockfalls, an accuracy of 0.5 inches (12.7mm) is usually adequate.

The purpose of the system is to indicate initial movement and/or to monitor acceleration (figure 5-9). Continued acceleration indicates a failure condition is developing. If rapid acceleration develops, consideration should be given to closing the highway until an assessment of potential danger can be made.

Figure 5-7. Multi-faced mirror reflector movement monitor for Electronic Distance Measurement of movement.

Figure 5-8. Electronic Distance Measuring unit with theodolite and in-unit computer. This unit should be set on a stable area to sight on reflecting mirrors.

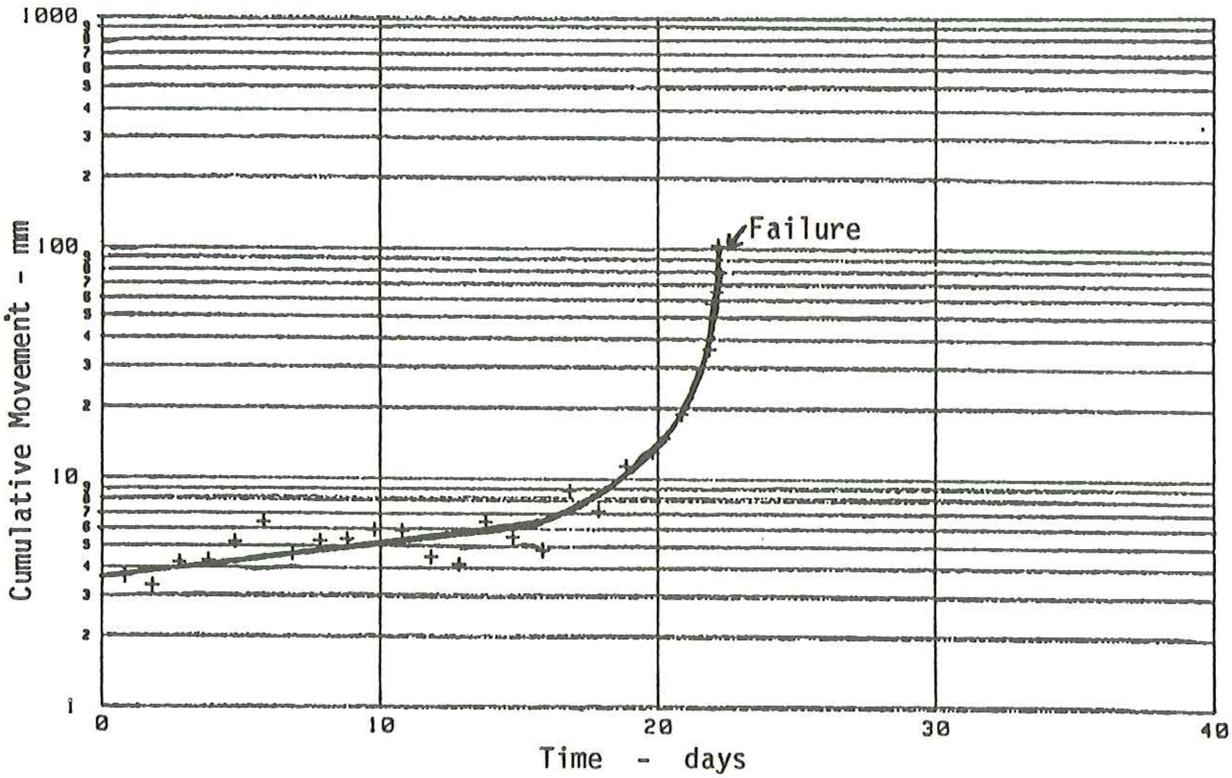


Figure 5-9. Plot of Cumulative Movement versus Time to evaluate acceleration of movement and reduction of stability. When the curve becomes vertical, upward failure will occur. Note the acceleration of movement after day 16 and the failure on day 22.

Weather conditions may influence readings. To correct for this influence at least one mirror on a known stable location should be measured during each cycle of readings.

The E.D.M. system is rapid, reliable and accurate.

5.2.3. Tilt Movement

Tilting of most rock movement will occur before failure.

SINCO has developed a portable tiltmeter (figure 5-10), which utilizes a closed loop force balanced servo-accelerator to measure tilt. Accuracy is quoted to be equivalent to a surface displacement of 0.06 inches (1.5mm) on a rock mass rotating about an axis 100 feet (30.5 meters) below the surface, or a displacement of 200 micro inches (5080 macro mm) over the 4-inch (101.6mm) length of the instrument.

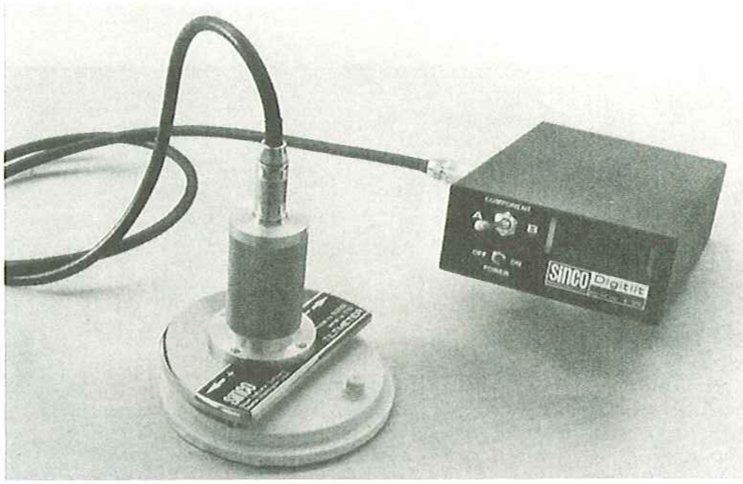


Figure 5-10. Tiltmeter to measure angular movement of rock. Several units are usually installed at one site (*Courtesy SINCO*).

Ceramic plates are cemented to the rock with one set of pegs lined in the most likely direction of movement. The type of movement (chapter 2) can be evaluated when multiple units are installed.

5.3 WARNING SYSTEMS

5.3.1. Extensometers

Movement across joints or cracks can be monitored with simple wire extensometers, sliding wire extensometers or electric strain meters. The sliding wire extensometer can be installed with a limit switch. The switch contains two separated copper contacts and a tension system. The spacing of the contacts is set to a predetermined distance at specific time periods. If movement exceeds the designated limit, a relay is tripped and warning lights and/or sirens are triggered (figure 5-11).

Electric strain meters (figure 5-12) can also be installed and developed as warning systems.

5.3.2. Fences

Warning fences to trigger warning signals have been used for decades by North American railways (figure 5-13). Where rockfalls, slides, or avalanches break a wire in an electrified fence above the track, warning signal lights on either side of the danger area are activated to signal the train of the danger. The signal is also transmitted to the dispatcher's office.

Figure 5-11. Light and siren connected to a wire line extensometer to warn of an excessive rate of movement.

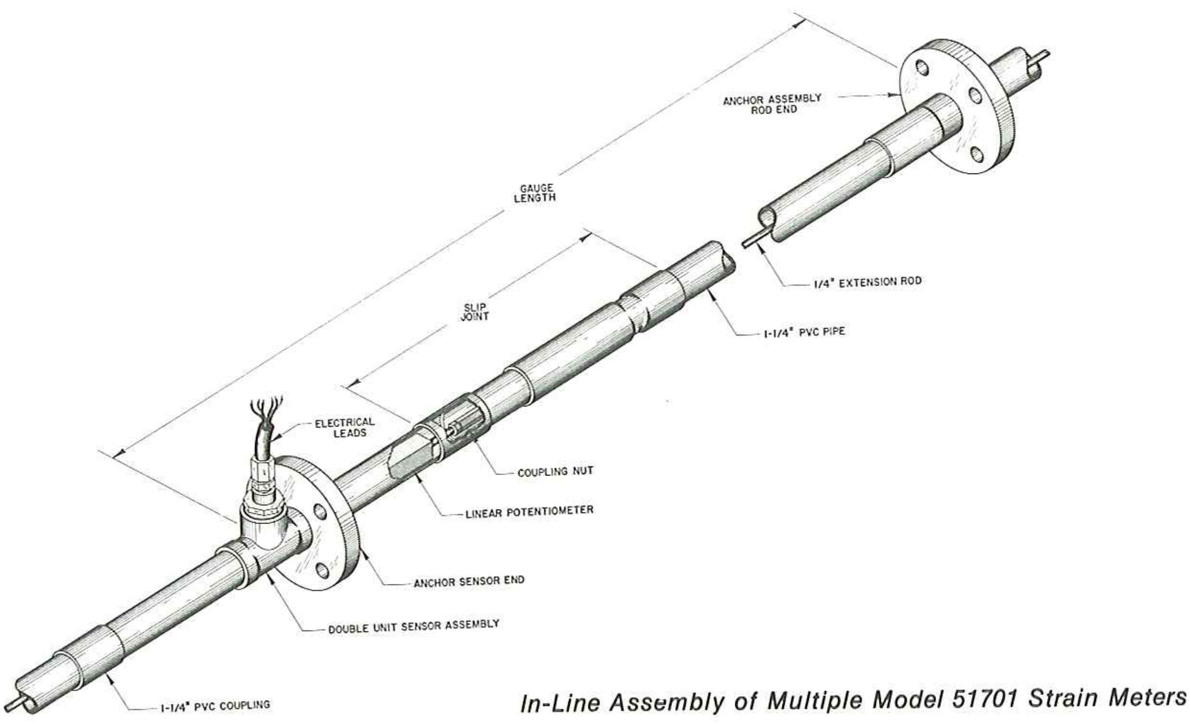


Figure 5-12. Electric Strain meter manufactured by SINCO. The unit measures movement between two fixed points.

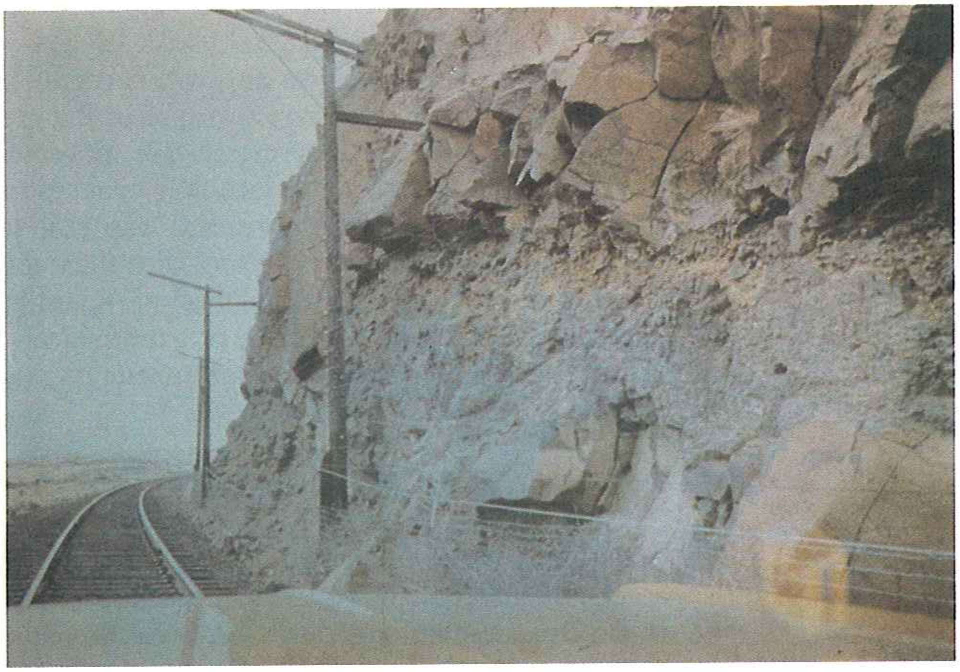


Figure 5-13. Multiple wire warning fence. Any rockfall that breaks a wire triggers a warning or excites a signal. Such a system can be used for highways where rockfall may come from a high elevation and move along a definite path.

This procedure has application on mountain highways where rockfall(s) may originate high above the highway and be confined to a single fall path. Breakage of the warning fence could trigger a signal so traffic crossarms would lower on either side of the rockfall path.

Warning fences are more applicable to highways with light-to-moderate traffic in higher mountain areas.

5.3.3. Future Systems

The Global Positioning System (GPS) which was developed by the military to monitor and position activities on the ground from multiple satellites, holds considerable promise to monitor movement in the future. Instrumentation now available to public agencies and companies can determine a location on the earth surface within about $\pm$164 feet (50$\pm$ meters). The military has developed the technology to an accuracy of about 0.5 inches (12.7mm). It is considered likely that special cooperation of the military could be obtained to use GPS monitoring in areas where rockfall stability is a major concern.

The system has the advantage of being unaffected by climatic conditions and can be operational 24 hours a day.

5.4. SUBSURFACE MOVEMENTS

Subsurface movement also will normally occur with rockfall movement. However, because of the greater ease and lesser expense of measuring surface movements, procedures to measure subsurface movement are not included in the manual. Any readers who wish to familiarize themselves with subsurface techniques are referred to the Geotechnical Engineering Investigation Manual by Roy E. Hunt (1984), or geotechnical instrumentation developers and manufacturers, such as SINCO.

5.5. BLAST MONITORING

Improper blasting that develops excessive seismic acceleration forces and excess gas pressure in discontinuities in the rock near the slope face can open discontinuities and new cracks up to 50 to 60 feet (15.3 to 18.3m) into the slope. The long term effect is to develop a rock face that will ravel and deteriorate for the life of the slope. This is undesirable.

It is very important to minimize blasting forces by using controlled blasting on any new or upgrading projects near the final face. Procedures are outlined in the FHWA Rock Blasting Manual.

Because blast design is not an exact science, a number of trial blasts should be established at the start of a project and when blasting conditions change. Where necessary, the trial blasts can be monitored to determine peak particle velocity. Numerous seismometers are available. The recently developed Blastmate Series II Seismograph by Instantel is a typical high quality unit. It is small, light-weight, computerized with printout, very robust, and with attachments, can monitor down boreholes, underwater, and air blasts (figure 5-14).

5.6. PATROLS

Experience has shown that the frequency of rockfalls increases substantially during and immediately following severe climatic events. As a result, increased inspection frequency by maintenance staff is advisable at such times that heavy rainfall, rapid snow melt, rapid water runoff and earthquakes occur.

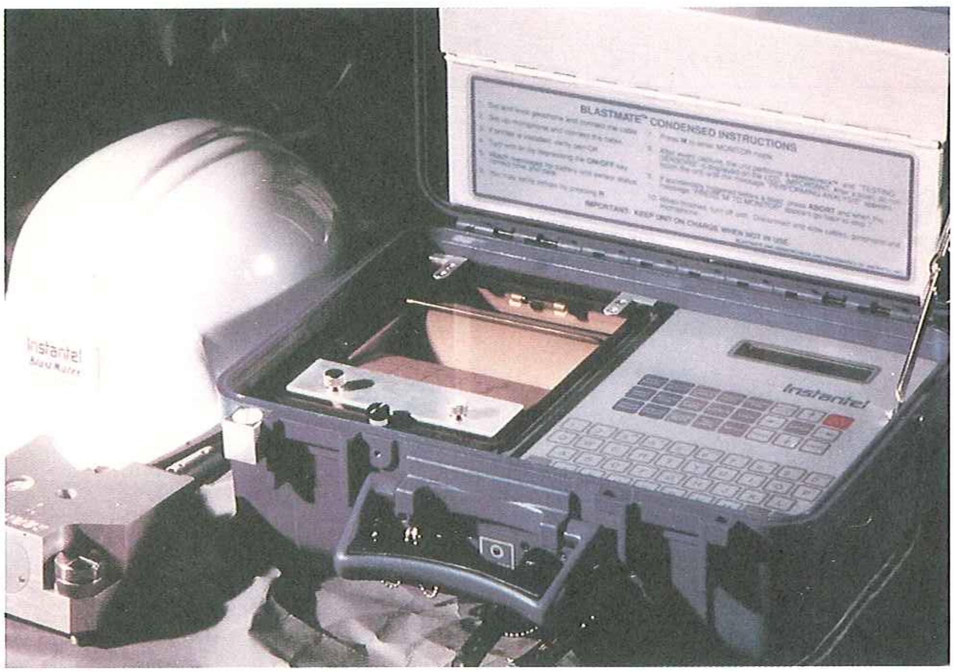


Figure 5-14. Light-weight Blastmate Series II Seismograph to monitor blasting (*Courtesy Instantel*).

CHAPTER 6

ROCKFALL ANALYSIS

6.1. INTRODUCTION

This section addresses the various methods used to analyze rockfall energies and rockfall trajectories. These methods include quantification of rockfall velocity, impact energy, and bounce heights. The three principal approaches to analysis (field testing, mathematical analysis, and empirical analysis) are discussed. All three approaches rely, to differing degrees, on actual field rock rolling data. Theoretical and computer analyses are used as tools to assist in rockfall control design. The analysis is inportant to determine the viabilty of barrier and protection mitigation measures.

6.2. SITE CHARACTERIZATION

6.2.1. Rock Behavior

Every rockfall site is different. Therefore, an analysis of the behavior of a rockfall event requires a thorough understanding of the site. This begins with investigating the history of rockfall at the site and understanding the characteristics of the site.

To begin the analysis, certain important questions need to be answered about the behavior of falling rock at the site.

- What is the nature of the rockfall event; is it composed of a single rock or a group of rocks falling?
- Is there a single source area or do the rocks originate from random locations on the slope face?
- As the rocks fall, do they hit other rocks and destabilize those rocks?
- What size rock typically reaches the base of the slope?
- Are the rocks rolling, bouncing, and/or sliding down the slope?

6.2.2. Rockfall Characteristics

Rockfall occurs as either individual rocks or a group of rocks. An individual rockfall event might be one rock falling or a single rock, falling and hitting other rocks, causing more rocks to fall individually. This could also result in a group of individual rocks falling within seconds of each other or one large group falling simultaneously. A group of rocks might also fall when an unstable rock cluster fails, which results in a group of rocks falling together as one mass. Each event would be analyzed differently.

The individual rocks should be analyzed independently with consideration given to two or three rocks that reach the runout zone simultaneously. Such an event typically might have low mass and high velocity. On the contrary, a larger, more massive group of falling rocks typically might have a lower velocity.

Much of the research performed in rockfall studies has dealt with individual rocks falling.

6.2.3. Historical Rockfall Events

In most cases, historical rockfall events reveal what rockfall events may be expected. An historical investigation should study rockfalls during "normal" years and "abnormal" years. A normal year might be when rainfall and weather patterns are average. Abnormal years would be when infrequent or rare events, such as "100 year" rainfall events or strong motion earthquakes occur.

It is important to differentiate between normal and abnormal conditions for the purpose of rockfall analysis. Risk analysis can then be used to determine which scenario warrants mitigation.

Traffic accident history can provide valuable information on the rockfall trajectory at the base of the slope. Verbal reports from people familiar with the area can provide useful information. Photos, eye-witness accounts, maintenance reports, and/or legal depositions might indicate the angle of rock impact. The impact angle would then be used to establish the rockfall trajectory for actual rockfall analysis. For example, records might reveal that the rock impacted a vehicle or the rock was hit by a vehicle after the rock came to rest on the roadway.

6.2.4. Size of the Rockfall

The size and shape of each fallen rock and potential rockfalls should be measured. First, the typical rock shape should be noted. Common shape descriptions are tabular, spherical, and disc shaped (figure 6-1). Second, the three principal rock axes ($\underline{x}$, $\underline{y}$, and $\underline{z}$) should be measured. Third, the specific gravity of the rock should be determined. Together this information is used to estimate the weight of the rocks and the moment of inertia of the falling rock body. This estimating procedure can be within 10 percent of the actual weight (Smith, Duffy, 1989). Fallen rocks at ground level can be weighed directly. A load cell attached to the bucket of a loader is a quick and easy method.

6.2.5. Rockfall Source and Runout/Impact Zone

The location from which rocks fall needs to be determined. This area is referred to as the source area. Additionally, the location where rocks stop needs to be determined. This area is referred to as the runout or impact zone. Knowing the beginning and final locations of the rockfall helps identify the path the rock travels. This information provides the rockfall height that is used to establish a cross-sectional location for the analysis. Also, from careful inspection of the rockfall path, information on the trajectory of the rockfall can be determined by identifying impact locations on the slope and at the base of the slope or observing where rocks have hit trees or similar tall obstructions.

Evaluating the size of rocks in the runout/impact zone and at the source can provide information about how the rocks break up during the rockfall. If large rocks are measured at the source but only fragments reach the runout zone, the rocks are breaking up either as they fall on the slope or at road level. If the former is true, then a smaller rock size can be used in the analysis, which can significantly impact analysis and mitigation measures.

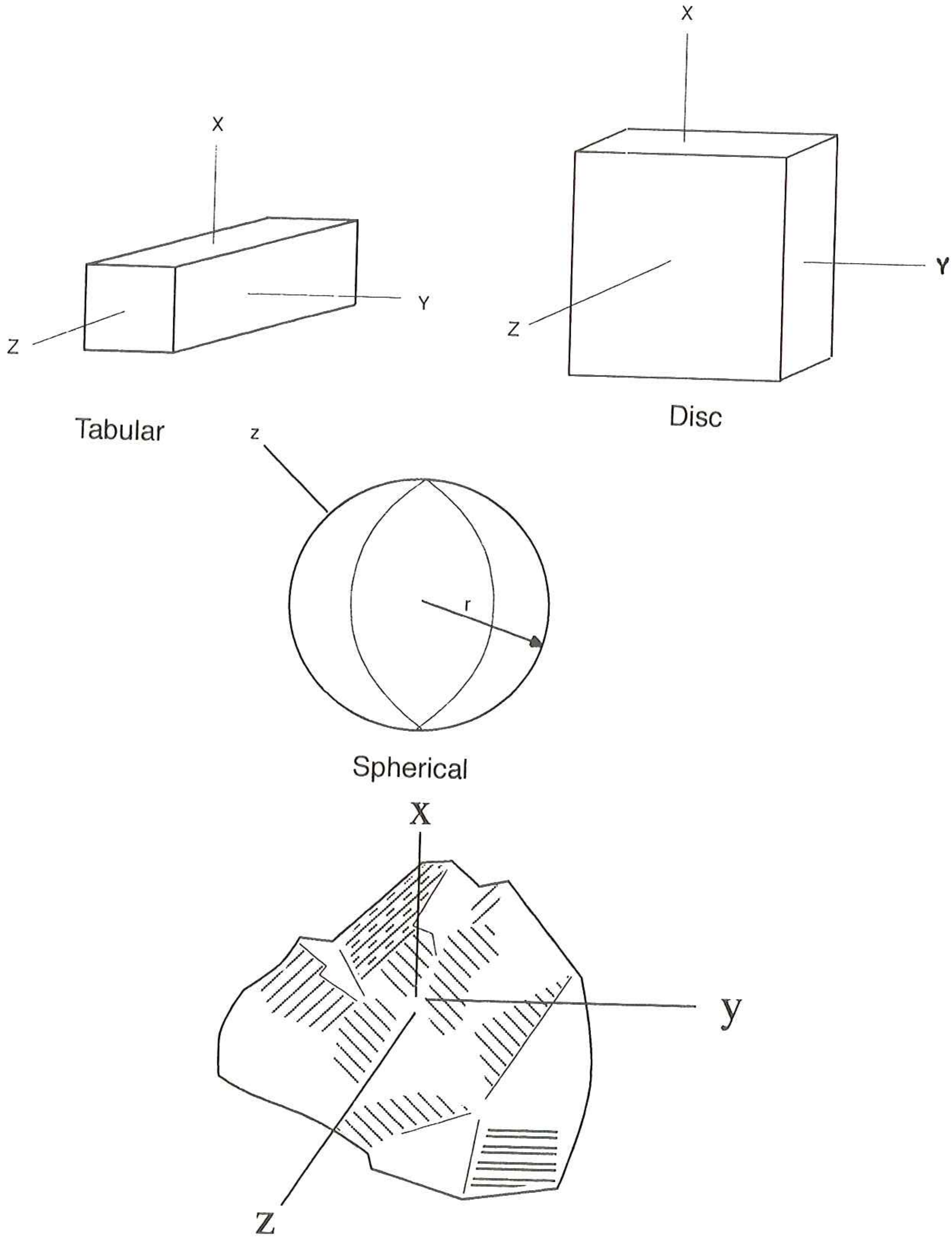


Figure 6-1. Common rockfall shapes tabular, disc and spherical.

6.3. SLOPE CHARACTERISTICS

Rockfall analysis requires a complete understanding of the slope. Each slope will be different and will have a variable slope angle, rock type, slope height, contour, soil cover, and vegetative cover. All of these features need to be characterized because they have an effect on the behavior of the rockfall event.

6.3.1. Cross Section

Each analysis should include a crosssection and contour map developed from a detailed survey of the slope. In some situations, survey points are located as close as 2 feet (.61 meters) apart. The crosssection should begin at the source area and end beyond the runout/impact zone. Significant changes in slope that could affect rockfall trajectories should be identified and recorded (figure 6-2). During this portion of the site investigation, surface roughness and vegetative cover are evaluated.

6.3.2. Surface Roughness

Slopes are described as smooth, irregular, or ragged. The condition of the slope in these terms is very important as it can significantly affect the behavior of a rolling rock. Most commonly, slope roughness is a measure of the slope irregularities relative to the size of the rock travelling down the slope (Pfeiffer and Higgins, 1990). The more significant this relationship, the rougher the slope. A rough slope can induce higher rock bouncing but, in some instances, rockfall velocities and impact energies may be reduced by a rough slope (figure 6-3). Features such as gullies, prominent rock outcrops, and rock ledges should be identified and measured.

6.3.3. Slope Cover

The effect of vegetation is important to each rockfall analysis. Vegetation usually reduces both velocities and bounce heights. In many cases, thick vegetation will stop a high percentage of rockfalls. Recent field studies in Switzerland indicate that approximately 60 percent of induced rockfalls collided with trees in a manner that stabilized the rocks (Zusammenfassung, 1985). Vegetation

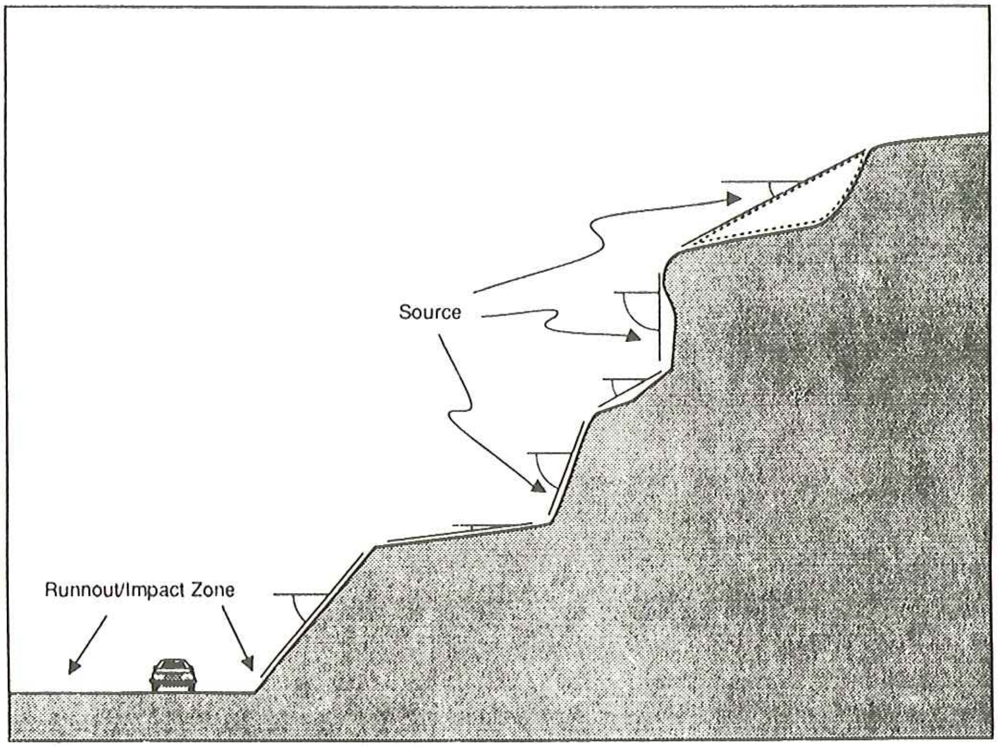


Figure 6-2. Cross section.

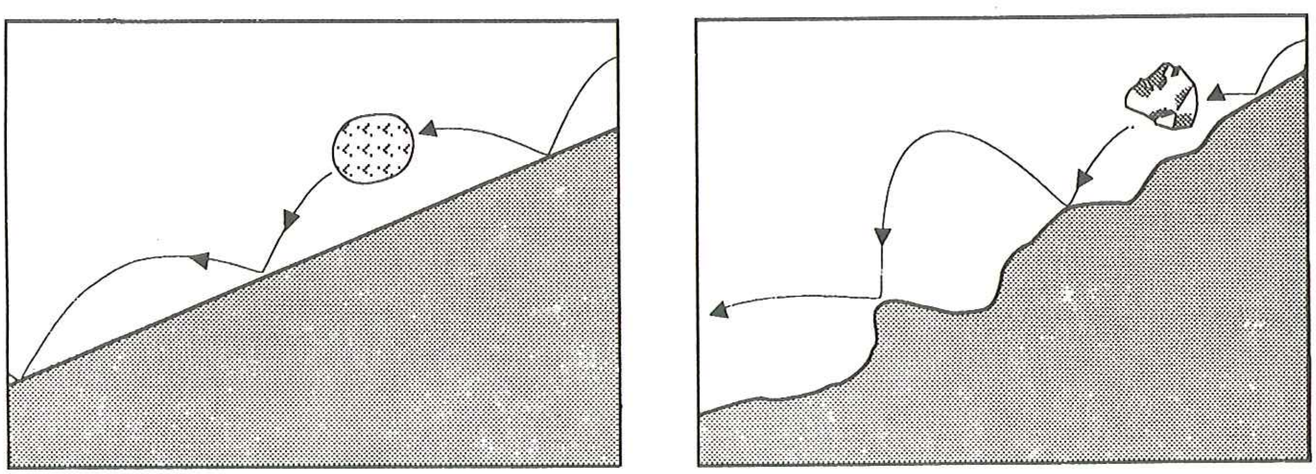


Figure 6-3. Surface roughness.

can also affect the path of the rockfall. However, vegetation can also cause rockfall because of the loosening and prying effect roots can have on rock blocks.

Soil cover also can affect rockfall trajectory. A thick layer of loose soil will dampen rockfall energy on impact, thereby reducing velocity, energy, and bounce heights. Therefore a thorough description of the soil cover (for example, thickness and density) should be recorded.

6.4. ROCKFALL ENERGY ANALYSIS

Rockfall impacts are commonly measured in terms of kilo-joules, foot-tons, or foot-pounds of total kinetic energy.

6.4.1. Kinetic Energy

Kinetic energy is the most common measurement used to describe rockfall for engineering design. Throughout the energy analysis, each rockfall is treated as a rigid body in motion. According to Chasles' theorem, any general displacement of a rigid body can be represented by a translation plus a rotation (Goldstein, 1950). Based on this theorem, the process of rockfall has two components: translational motion and rotational motion. These two components can be quantified as energy in motion or kinetic energy. Calculation of these kinetic energies is based on the assumption that the mass of the rock is concentrated at the center of mass and its motion revolves around the center of mass. Therfore, rockfall motion is the sum of the translational kinetic energy (KE_T) and the angular kinetic energy (KE_A). This sum, the total kinetic energy (KE), is expressed mathematically as:

$$\text{Total KE} = KE_T + KE_A = 1/2mv^2 + 1/2I\omega^2$$

where <u>m</u> is the mass of the rock, <u>v</u> is the velocity of the rock just before impact, <u>I</u> is the moment of inertia of the rock as it spins, and ω is the angular velocity of the spinning rock just before impact.

6.4.2. Mass and Weight

The weight (W) of the body is the gravitational force with which the earth attracts the body. Mass (m) is the property a body has of resisting any change in its state of rest and is a measure of inertia of the rock body. When working in English units, the rock mass (m) is calculated by dividing the rock's weight (W) by the acceleration due to gravity (g). In metric units, the acceleration of gravity (g) is included in the unit of measure for weight.

$$m = {}^{\omega}/_{g} \quad \text{(For SI Units)}$$

As stated earlier, an estimate of the weight of the rock should be made by measuring the three principal rock axes (x, y and z). These values are used to calculate a representative volume (V) of the boulder. The rock weight equals rock volume (V) multiplied by the unit weight of the rock. The unit weight is determined from field samples tested in the laboratory for specific gravity (SG) that, when multiplied by the unit weight of water, equals the unit weight of the rock.

$$(SG_{rock})(\gamma_{water}) = (\gamma_{rock})$$

6.4.3. Velocity

Translational velocity and angular velocity must be quantified to determine total kinetic energy. Translational velocity (v) is the velocity of the rock mass concentrated at the center of the rock body. This velocity is determined by measuring the time (t) it takes the rock to travel some known distance (d).

$$\underline{v} = \text{distance} / \text{time}$$

Angular velocity (*w*) is the velocity of the rock mass spinning around the center of the rock body. Angular velocity (ω) is determined by measuring the time it takes a rock to complete one revolution (360 degrees = π radians).

$$\omega = \text{radians} / \text{time}$$

This information is obtained from video tapes and slow motion film footage of induced rockfall events. With visible reference lines on the slope and film editing equipment capable of achieving frame-by-frame control, accurate measurements of the time (t) it takes for the rocks to travel between reference lines and the time it takes a rock to spin one revolution can be obtained.

6.4.4. Moment of Inertia

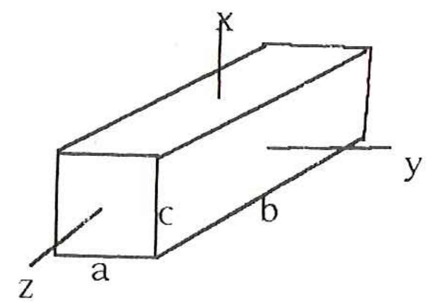
The moment of inertia (I) depends on the mass distribution relative to the axis of rotation of the rock body (Tipler, 1976). The value of the moment of inertia of a rock body about a particular axis of rotation depends not only upon the body's mass, but also upon how the mass is distributed about the axis. In rockfall analysis, the axes are typically assumed to be at the center mass of the rock body. The same principal axes (x, y and z) used to estimate boulder weight are used in inertia calculations. For these calculations, equations are selected to represent rectangular bodies and spherical bodies, and the boulders are assumed to be homogeneous solids.

The motion of rectangular bodies is a function of the axis about which they rotate. In rockfall analysis, rotation is assumed to occur around only one of the three principal axes (x, y or z) and is described by three equations (figure 6-4). The motion of the spherical bodies is a function of the radius of the bodies and is described by a single equation (figure 6-5). The axes of rotation are determined from videos and also motion films.

In many cases, it has been observed that rocks revolve around the longest axis for about the first 150 feet (45.8 meters). As rock velocity increases, rocks then revolve around the shortest axis (Smith, Duffy 1989). In these cases, the longest dimensions should be used in angular KE calculations.

6.5. ROCKFALL TRAJECTORY ANALYSIS

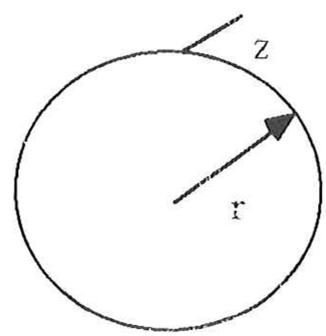
There are three principal methods of analyzing rockfall trajectories. One is to perform field test whereby rocks are rolled and the behavior of the falling rock is observed. The second is to construct a mathematical model. This is typically done using the various computer models developed for that purpose. The third is to rely on empirical data that characterize the behavior of rockfall for different slope characteristics.



where m = mass

$$I_{xx} = 1/12\ m\ (a^2 + b^2)$$
$$I_{yy} = 1/12\ m\ (c^2 + b^2)$$
$$I_{zz} = 1/12\ m\ (a^2 + c^2)$$

Figure 6-4. Rectangular parallelepiped.



where m = mass

$$I_{zz} = 2/5\ mr^2$$

Figure 6-5. Sphere.

6.5.1. Field Test Analysis

Whenever possible, rolling rocks at the site will provide the most accurate information for rockfall analysis. However, this is not always possible due to safety considerations and economic constraints. Along transportation corridors, sometimes traffic can be slowed several miles away to allow enough time to roll 10 to 15 rocks. This technique works best if the test area can be prepared without impacting the travelled way. An alternative is to conduct the rock-rolling tests on a similar slope of similar rock type and characteristics that is away from the road or on a road with less traffic.

Obtaining good test data for rockfall analysis requires careful preparation. Measurements of the rocks to be rolled and a properly prepared test slope are needed. Most importantly, film and video equipment should be in place and operational.

6.5.2. Test Rocks

When testing in the field, source rocks will usually be scattered. However, some locations may have a rock pile source. Often times, rock rolling can be performed in conjunction with scaling operations. In either case, each rock must be located on the slope, measured and marked before rolling. Frequently, rocks will break up as they fall. Many rockfall research projects have obtained test rocks from a local stockpile, quarries or from the hillside. Whatever the rock source, a description of the rock and its specific gravity is required. The rocks should also be measured and weighed after rolling to determine their weight upon impact.

Prior to rock rolling, the three principal rock axes are measured. These values are used to estimate rock weight and inertia. Where possible, rocks can be accurately weighed with a load cell. This will always be possible at road level. Actual weights can be compared to estimated weights to evaluate estimated weight accuracy and rock breakage.

Record the method of initiating the rockfall. Possible methods include prying rocks off the slope by hand, pulling rocks with a cable, or using heavy equipment to drop rocks. This information is important to determine the initial velocity for rockfall analysis .

Recent research has shown that rockfall diameters of 2 feet (0.6 meters), when falling at high velocities, cause a bullet effect. This can render certain mitigation measures useless unless this occurrence is considered.

6.5.3. Test Slope

To fully utilize the film and video footage, place reference lines on the slope and in the runout/impact perpendicular to the slope axis (figure 6-6). This allows detailed measurements of rock travel time over a known distance. This information is used to calculate rockfall velocities. Yellow, three-inch wide "Caution" tape works well because of the high visibility of the tape.

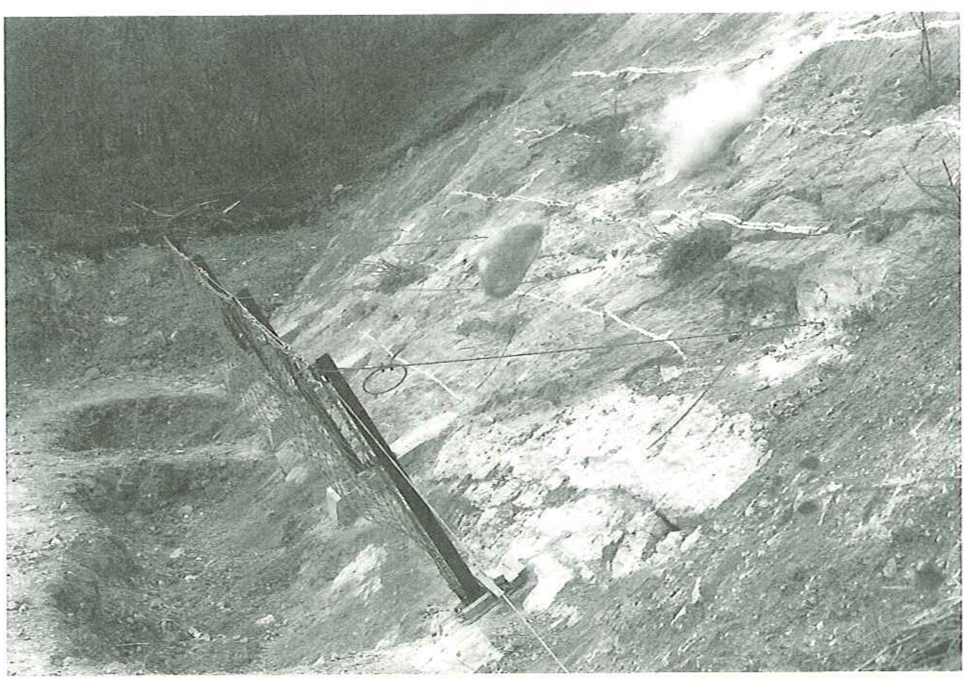


Figure 6-6. Photo showing sample reference line layout during rock rolling tests *(Courtesy Brugg Cable Products).*

Typical reference-line spacing is 10-foot (3.1 meters) intervals for a distance of 50 feet (15.3 meters) upslope from the runout/impact area, followed by larger intervals marked up to the launching point. Below the runout/impact zone, reference lines might be marked on the ground at 10-foot (3.1 meters) intervals through the runout/impact zone. Slope geometry will cause spacing to vary. Narrowing spacing is recommended because it will provide greater detail.

In addition, depending on the slope, stadia rods 3 feet to 6 feet (.9 to 1.8 meters) in length should be placed on the slope for bounce height analysis. Often the rods are placed randomly because of slope constraints, but an effort should be made to place rods at or near the reference lines.

These procedures do not apply to very steep slopes.

6.5.4. Data Collection

Videotape and film footage are used to collect the data for the rockfall trajectory analysis. Rock rolling has to be recorded on video and/or high-speed (16 mm) film from a minimum of two different camera views, but four or more camera views are preferred. The multiple cameras will show more detail from different angles and also serve as backups if a camera fails.

American video equipment records at 30 frames per second while most foreign equipment records at 25 frames per second. Slow-motion coverage is recorded on high-speed film (60 to 80 frames per second).

The recommended four cameras should capture two side views, one oblique view, and one front view (figure 6-7). At a minimum, one oblique view of the rock falling should be recorded. Recommended camera angles are a sweep camera following the rock down the slope with a minimum of 50 feet of slope surrounding the rockfall in the field of view. The other cameras may be stationary, focusing on side views and front views of the slope face. It is *very important* to obtain high-quality film and video of the tests. Without this information, the tests cannot be analyzed. Poor-quality coverage might miss important details. All data should be recorded on a data sheet for each rock roll (figure 6-8).

6.6. MATHEMATICAL ANALYSIS

6.6.1. Computer Modelling

Computer modelling can be used as a tool to study the behavior of rockfalls, determine the need for mitigation, and aid in design (Pfeiffer and Higgins, 1990). Modelling should be used in conjunction with field data and good judgment. Computer modelling allows designers and investigators to observe dozens or even hundreds of simulated rockfall events.

Through the years, several computer programs designed to model rockfalls have been developed in the United States (Evans, 1989). In 1985, the North Carolina Department of Transportation developed a rockfall simulation model. In 1987, Evert Hoek of Golder Associates wrote a computer program to model rockfall. The Colorado Department of Highways completed its own rockfall simulation program in 1988. This model is being updated periadically. Contact the Colorado Department of Transportation for the latest version. Richard Call of Call & Nicholas, Inc., developed a rockfall simulation in 1989. Outside the United States, numerous programs have been developed in Canada, Germany, Italy, France, Switzerland and Spain (Haller, 1993) (table 6-1).

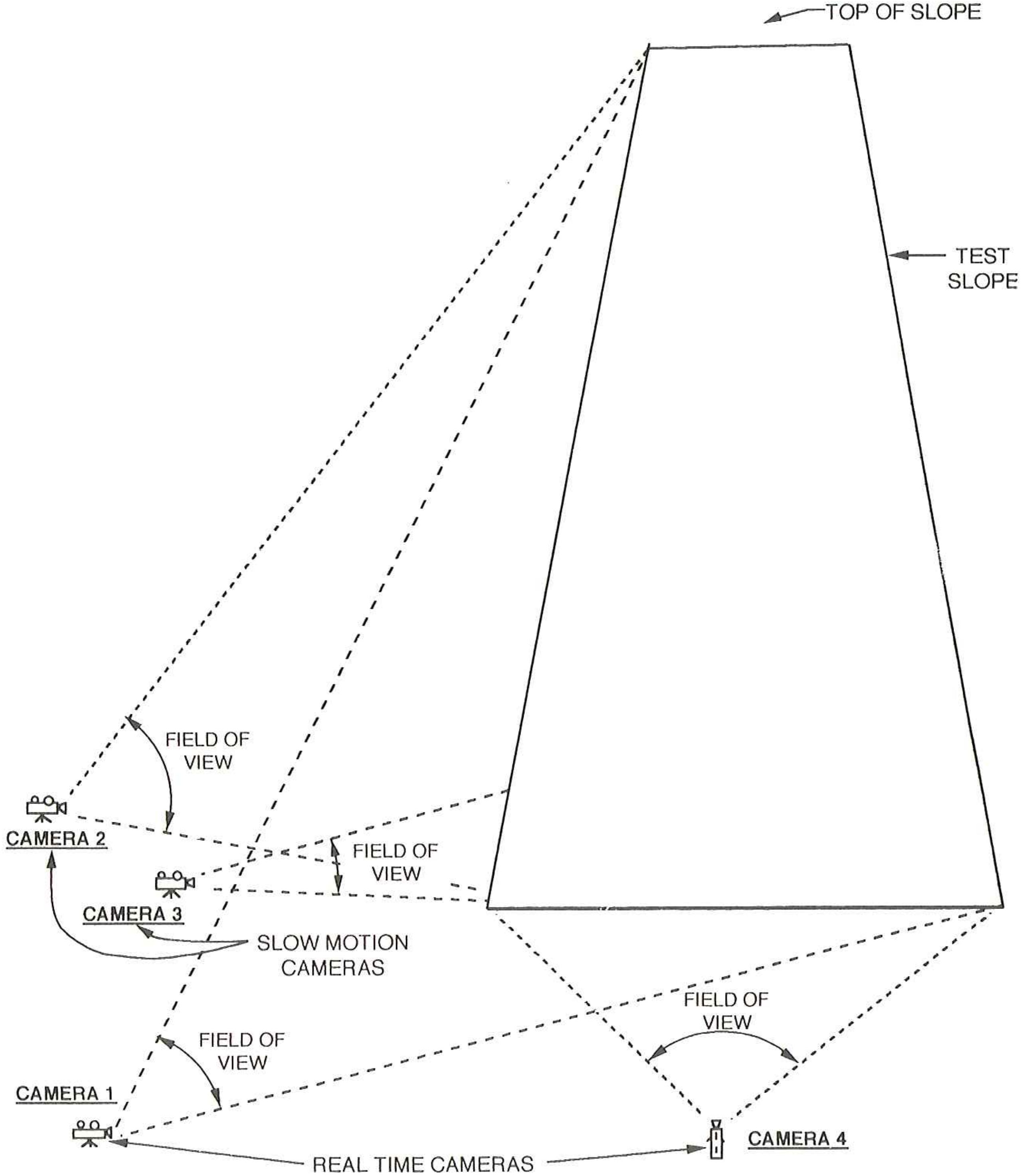


Figure 6-7. Sample camera locations for filming rock rolls.

DESIGN NO. ________ DATE ____________________
ROLL NO. ________ ROCK I.D. ________

A. ROCKFALL DATA

1. DIMENSIONS:________ METERS(m) ________FEET (ft)
2. VOLUME(V):________ m3 ________ ft3
3. UNIT WEIGHT( ): ________ Kg/m3 ________ Lb/ft3
4. WEIGHT (W): ________ Kg ________ Lb
5. MODE OF TRAVEL BEFORE IMPACT (roll, bounce, slide, freefall, other):
____

ESTIMATED WEIGHT:1. ________ ACTUAL WEIGHT: ________
2. ________
3. ________ % ERROR: ________

B. MOMENT OF INERTIA (I)

1. SHAPE: ________ SKETCH:
2. AXIS OF ROTATION: ________
3. I = ________

C. VELOCITY (v)

1. TRANSLATIONAL
   a. LENGTH OF ROLL IN TIME = t ______m ______ft
   b. TIME (t) ________ seconds
   c. VELOCITY (v): ________ m/sec
   ________ ft/sec

2. ANGULAR
   a. ROTATION: ____ degrees
   ____ radians
   b. TIME: ________ seconds
   c. ANGULAR VELOCITY ( ):
   ______ sec-1 (radians)

D. TOTAL KINETIC ENERGY (K.E.) = TRANSLATIONAL K.E. + ANGULAR K.E.

1. TRANSLATIONAL K.E. = 1/2 mv2
________ Kilojoules ________ Foot-tons

2. ANGULAR K.E. = 1/2I 2
________ Kilojoules ________ Foot-tons

3. TOTAL K.E. = TRANSLATIONAL K.E. + ANGULAR K.E.
________ Kilojoules ________ Foot-tons

E. REMARKS

Figure 6-8. Example of a rock rolling test data sheet.

Table 6-1. Computer Models

C. Azimi, and P. Desvarreux, Calcul de Chutes de Blocs et Verification sur Modele Reduit, Rap. ADRGT, JUIN 1977.

D. Bozzolo, R. Pamini, Modello Matematico per lo Studio Della Caduta dei Massi, Laboratorio di Fisico Terrestre-ICTS, Lugano-Trevano, Switzerland, 1982.

Richard D. Call and T.M. Ryan, Computer Program BENCH, (Catch Bench Geometry Based on the Ritchie Model), Call & Nicholas Inc., Tucson, AZ, 1988.

D'Appolonia Consulting Engineers, Inc. Rockfall Analysis, North Carolina Department of Transportation and Highway Safety Report, Raleigh, NC, 1979.

E. Hoek, Rockfall-A Program in Basic for the Analysis of Rockfalls from Slopes, Golder Associates, Vancouver, B.C., 1987.

Y. Kobayashi, E.L. Harp, and T. Kagawa, 1990 "Simulation of Rockfalls Triggered by Earthquakes," Rock Mechanics and Rock Engineer 23, pp 1-20.

T.J. Pfeiffer, and J.A Higgins, "Rockfall Hazard Analysis Using the Colorado Rockfall Simulation Program," TRB, Transportation Research Board, 1990.

D.R. Piteau, and R. Clayton, Discussion of paper, Computerization Design of Rock Slopes Using Interactive Graphics for the Input and Output of Geometrical Data, by P.A. Cundall, M.D. Voegele, and C. Fairhurst, In Design Methods in Rock Mechanics (Fairhurst and Crouch) 16th. Symposium of Rock Mechanics, ASCE, pp 62-63, 1997.

Raymond Spang, Protection Against Rockfall-Stepchild in the Design of Rock Slopes, 6th International Congress on Rock Mechanics, Montreal, Canada, pp. 551-557. (Geotechnical Consultants, Witten, Germany), 1987.

In the United States, the Colorado Rockfall Simulation Program (CRSP) is the most widely used program. Colorado's program has proven to be the most consistent in predicting rockfall behavior on differing test slopes (Evans, 1989). CRSP is the principal program discussed in this section. However, use, refinement, and development of other programs is encouraged.

6.6.2. Colorado Rockfall Simulation Program (CRSP)

CRSP was developed to model rockfall behavior and to provide statistical analysis of probable rockfall events at a given site (Pfeiffer, 1989). The program is based upon the principles of physics that apply equations of gravitational acceleration and conservation of energy to describe a body in motion. CRSP is based upon field observations and studies of actual rockfalls.

A. *Input*

The program relies on six principal input variables: a detailed cross section, surface roughness, surface cover, surface hardness, rock size, and rock shape.

The program also allows input of a specific location to be analyzed. This location is typically selected where protective measures are proposed or where the area to be protected is known.

Cross sections of the slope should highlight the major slope changes that could affect a falling rock. These changes are typically in degrees of slope angle for a distance of several rockfall diameters. The slope is categorized into different cells that represent changes in slope angles, roughness, hardness, and cover.

Surface roughness is a measure of the raggedness of the slope in relationship to the rockfall diameter. If all slopes were smooth, and perfect spheres were rolled down the slope's face, modelling would be very accurate. However, this condition is never fully realized in nature. Most slopes are irregular and variable with ledges and outcroppings that affect rockfall trajectory (bouncing, rolling, or sliding), which is also related to the diameter and shape of the falling rock. This program calculates a ratio relating the size of the rock to the slope roughness. This is a difficult number to obtain without a detailed look at the slope. Typically this is done while surveying the cross section in the field.

Surface cover addresses characteristics of the surface material within each cell and is referred to as the tangential coefficient (Rt). This includes soil and vegetative cover (table 6-2), which have an effect on the behavior of the falling rock by absorbing energy and may stop the rockfall.

Surface hardness characterizes the hardness of the surface rock and the slope surface and is referred to as the normal coefficient (Rn). This feature will affect the energy-absorbing qualities of the slope surface (table 6-3). A bare rock surface will cause greater bouncing while a deep soil cover will absorb considerable energy, possibly reducing bouncing.

Rock size is given in diameter or longest and shortest axes, depending on which shape best describes the rock. Rock size and rock shape can affect the trajectory. As stated previously, the available shapes are spherical, tabular, and disk.

When possible, field rock rolling tests should be performed in conjunction with the modelling program. Modelling actual rockfall trajectories from field tests will require determining, by trial and error, site specific slope coefficients that should be used in the modelling analysis.

B. Output

The program provides statistical estimates of probable rockfall velocities and bounce height statistics at various locations on the slope. The program defaults to 100 rolls, but any number is possible.

On screen, a cross section is displayed with the trajectories of each individual simulation. This is followed by an output of statistical data on the average and maximum bounce heights and velocities along the entire slope and specific data on the bounce height velocity and energy at a specific predetermined analysis location (figures 6-9 and 6-10). This information can be used to determine a typical rockfall trajectory that could be used to identify areas of low bounce heights and low energy. Knowledge of such areas is

Table 6-2. Suggested Tangential Coefficient Input Values.

Tangential Coefficient Rt	Description of Slope
0.87 - 0.92	Smooth hard surfaces,such as pavement or smooth bedrock surfaces.
0.83 - 0.87	Most bedrock surfaces and talus with no vegetation.
0.82 - 0.85	Most talus slopes with low vegetation.
0.80 - 0.83	Vegetated talus slopes and soil slopes with scarce vegetation.
0.78 - 0.82	Brush covered soil slope.

Table 6-3. Suggested Normal Coefficient Input Values.

Normal Coefficient Rn	Description of Slope
0.37 - 0.42	Smooth hard surfaces and paving.
0.33 - 0.37	Most bedrock and boulder fields.
0.30 - 0.33	Talus and firm soil slopes.
0.28 - 0.30	Soft soil slopes.

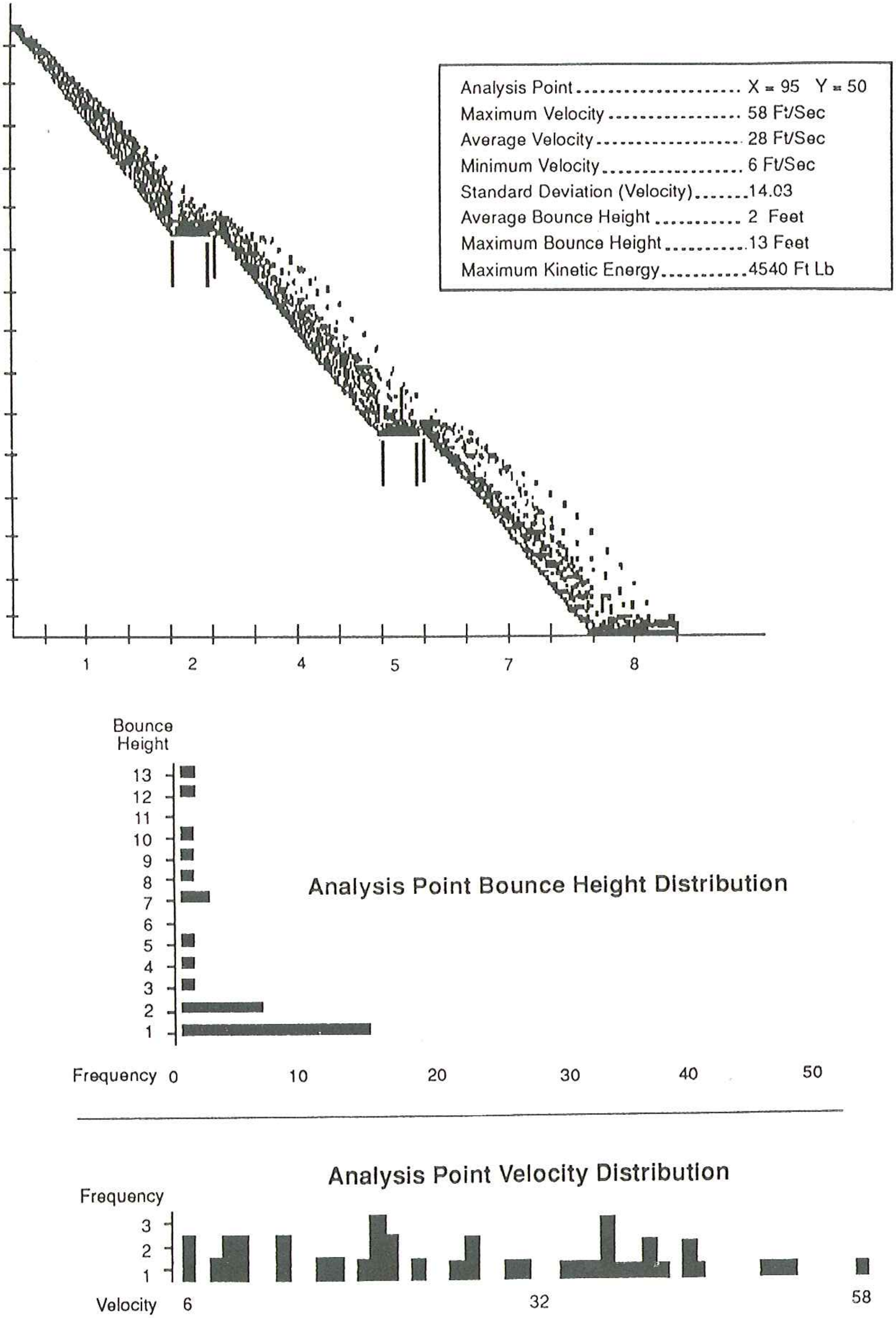
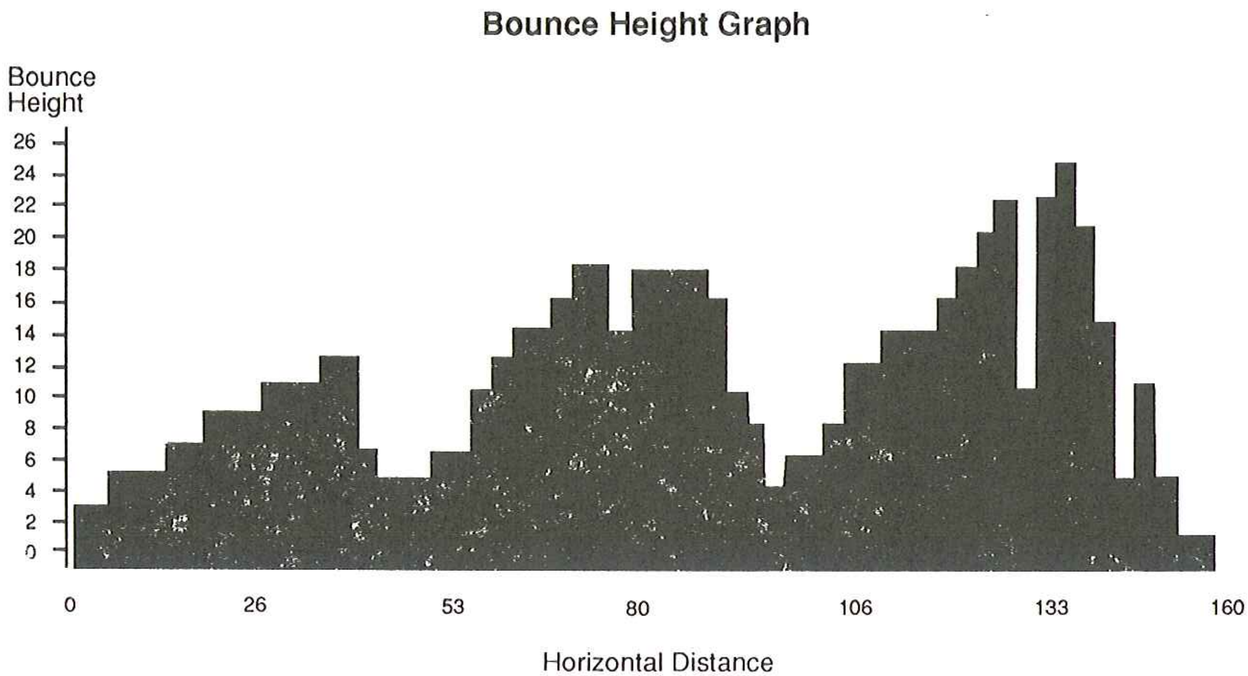


Figure 6-9. Cross section and trajectory with analysis point data from CRSP output.



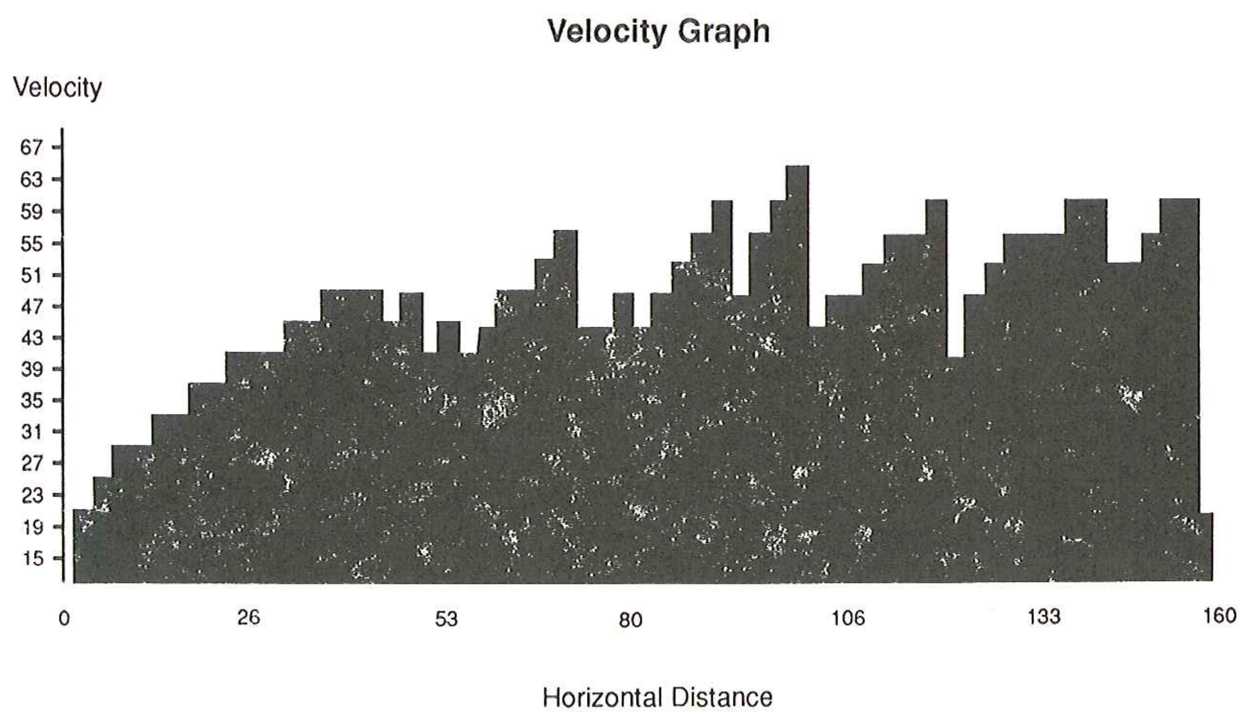


Figure 6-10. Statistical data from CRSP output on average velocity and bounce heights along the entire slope.

useful in optimizing appropriate mitigation measures. CRSP also is used to determine energy and bounce heights at a particular location and to assist in the determination of the barrier design (post section and spacing, mesh size, and wire or cable strength and fence height and location).

6.7. PERFORMANCE AND EVALUATION

Performance is evaluated by comparing the relationships between impact loading, maintenance, and efficiency. Impact loading is simply the amount of energy hitting the barrier. Maintenance indicated repair necessary at the various impact loads. Efficiency represents the importance of impact location. A chart has been developed for five common flexible barriers in use today: chain link fence, flex post fence, Oregon/Washington hanging fence, wire rope nets and ring nets (figure 6-12). Colorado's geosynthetic wall is also included.

The black area in figure 6-11 represents the design load limit. Within this range, efficiency is optimum and maintenance is minimum. Above this range, efficiency decreases and maintenance increases. Impacts within the shaded range will be stopped but damage could be significant. Beyond the shaded area, the nets are not effective for design purposes. At this energy level the rocks will not be stopped, but rockfall energy will be attenuated.

The ideal impact for any flexible barrier is a center-net impact. A center impact allows the barrier to fully flex, thereby efficiently absorbing energy with minimal damage. The barrier flexibility increases the time for the rock to decelerate, therefore decreasing the total force on the system and increasing the barrier's ability to absorb high energies with minimal maintenance.

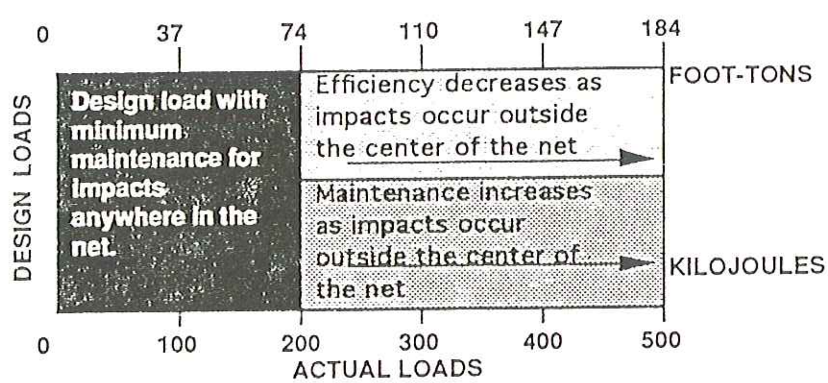


Figure 6-11. Explanation of design load chart.

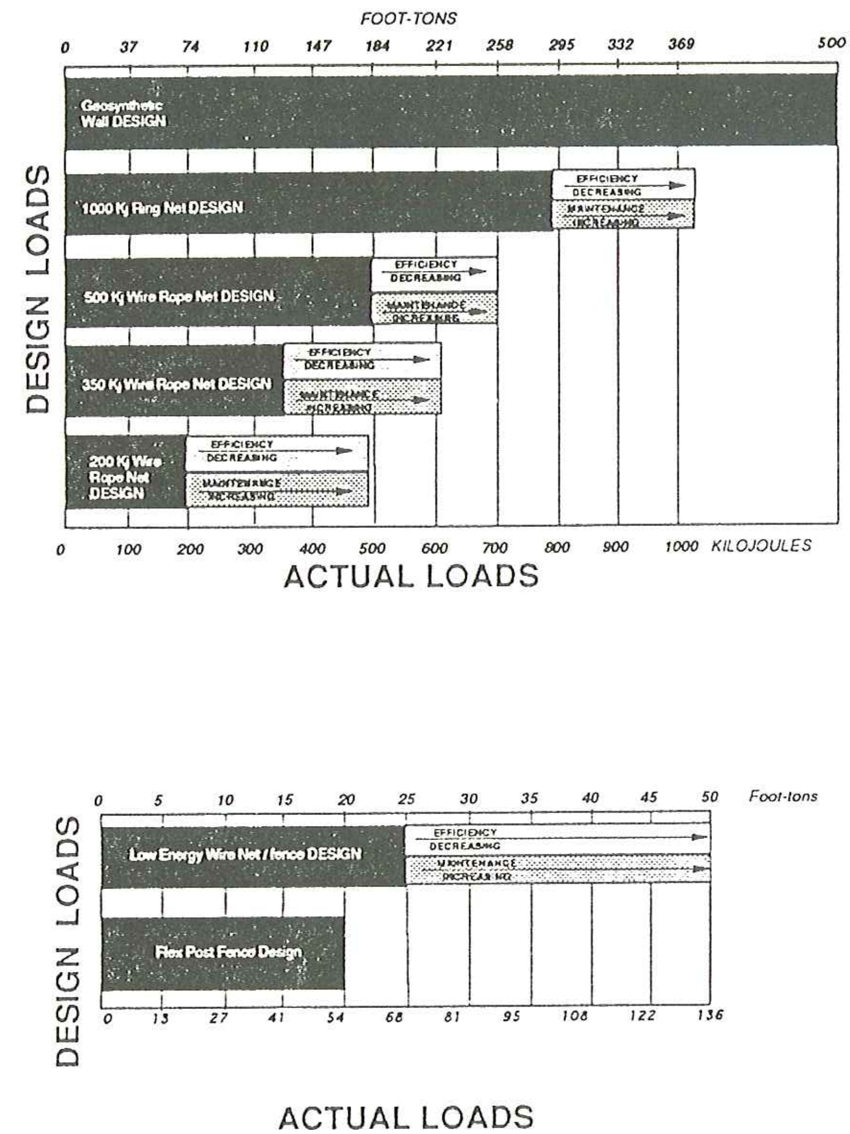


Figure 6-12. Design load for various flexible barriers.

6.8. EMPIRICAL ANALYSIS

Over the years, researchers have developed tables and charts that describe the behavior of rockfall based on basic slope parameters (for example, slope angle and slope heights). This information is commonly presented based on the results of rock rolls from actual field tests.

Numerous researchers have rolled rocks on slopes and recorded the trajectories (table 6-4). Much of these data have been presented in tables and charts so that designers and investigators could use this information to predict rockfall trajectories at other sites.

6.8.1 Ritchie Criteria

The most well-known and widely used empirical data is the Ritchie Criteria (Ritchie, 1963), based on Arthur M. Ritchie's rockfall research conducted in the 1960s. His procedure was to roll rocks on various slopes of differing angles and heights and record the rocks' trajectories. Data were then collected on the mode of the fall (rolling, bouncing, sliding, or free falling) and on locations where rocks landed and stopped.

Based upon this research, the original Ritchie Criteria were developed. These criteria present rockfall catchment ditch geometries that prevent most rocks from free falling or rolling onto the travelled way. These recently updated criteria are based on the slope height and slope angle (figure 6-11).

Vertical slopes will be stable where the structural geology is favorable and the rock strength exceeds about 1000 psi. The adjacent ditch should have adequate width to clean out with a front end loader. This reduces the potential of rockfall on to the travelled roadway and reduces quantities for excavation.

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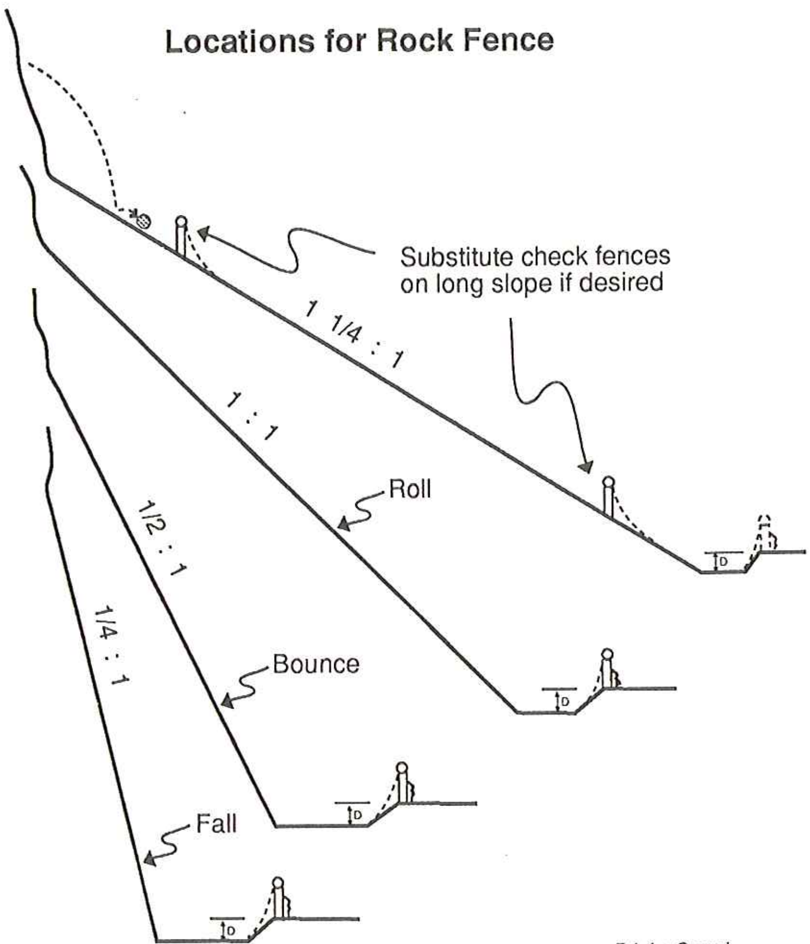


H	W	D
ROCK SLOPE : NEAR VERTICAL		
15 - 30	10	3
30 - 60	15	4
over 60	20	4
ROCK SLOPE: 0.25 : 1 & 0.3 : 1		
15 - 30	10	3
30 - 60	15	4
60 - 100	20	6 F
over 100	25	6 F
ROCK SLOPE : 0.5 : 1		
15 - 30	10	4
30 - 60	15	6 F
60 - 100	20	6 F
over 100	25	8 F
ROCK SLOPE : 0.75 : 1		
0 - 30	10	3
30 - 60	15	4
over 60	15	6 F
ROCK SLOPE : 1:1		
0 - 30	10	3
30 - 60	10	5 F
over 60	15	6 F
ROCK SLOPE : 1.25 : 1		
Use rock fence on shoulder		

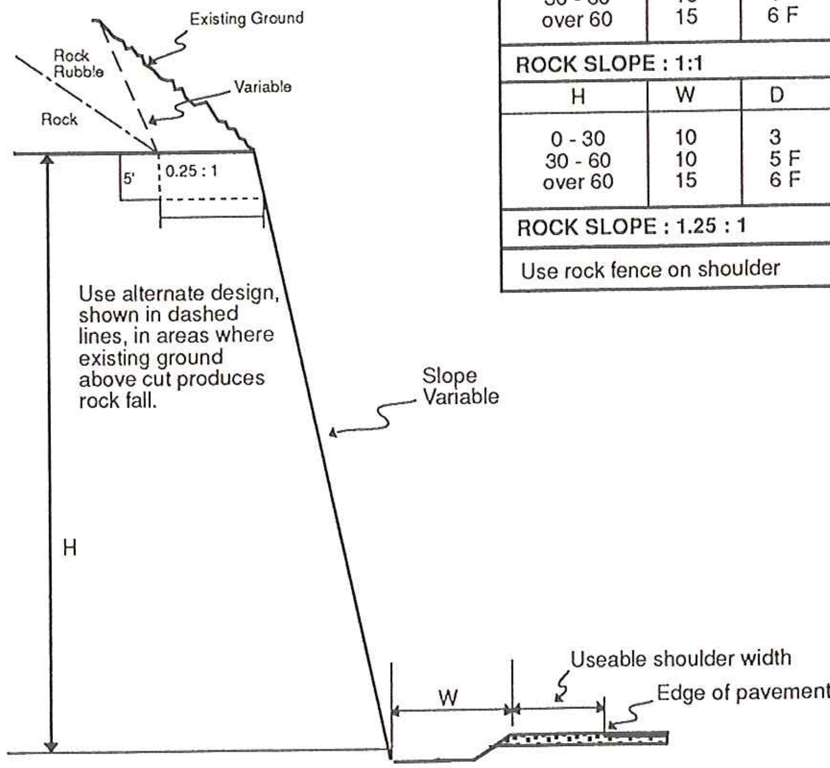


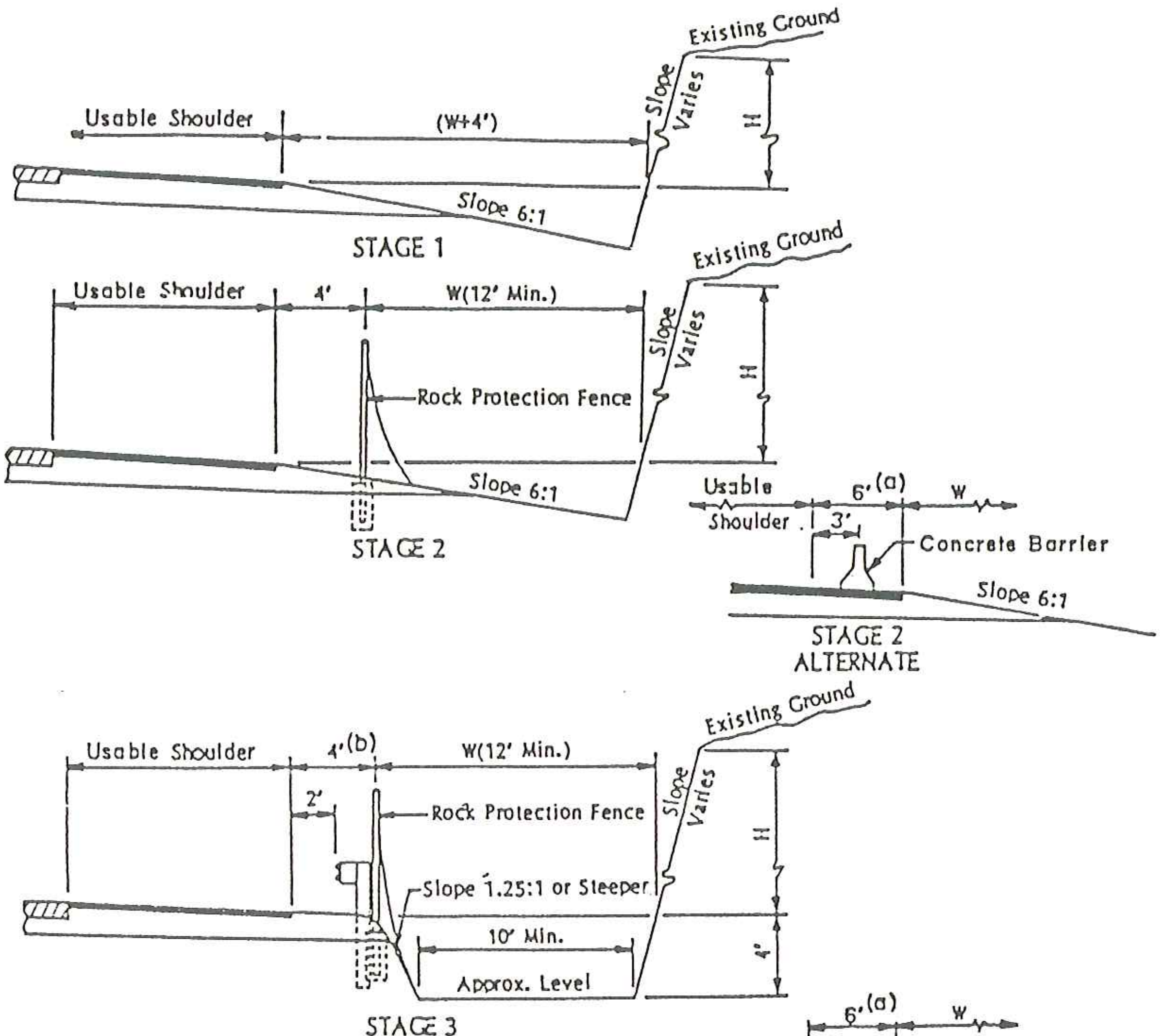
Figure 6-13. Modified Ritchie Criteria. Where structural geology is favorable, vertical slopes recommended. Further modification has been developed by the Washington State DOT (figures 6-14 and 6-15).

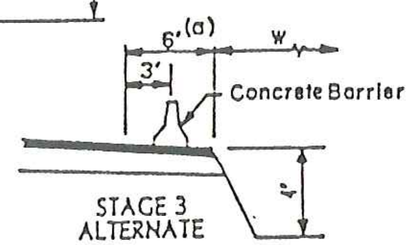
Where jointed rock exists, the slope design can be based on nonlinear strength envelopes using the CSIR geomechanics classification and Hoek and Brown Criteria (Hoek and Brown, 1980).

6.8.2. Discussion

All of the methods presented in this section can be very useful tools for analysis when used properly. The most accurate analysis will use all three methods during the site analysis. Each method can be used to verify or disqualify the others. The result will be an accurate picture of rockfall behavior and the best information available to design rockfall mitigation procedures at the site.

The Washington State DOT has modified the original Ritchie Criteria. Current design criteria, excerpted from the Washington State DOT Roadway Design Manual are presented in figures 6-14 and 6-15. Design A (figure 6-14) applies to rock cuts not designated as talus slope conditions. Design B (figure 6-15) applies to talus slopes.





ROCK SL.	H(FT.)	W(FT.)
NEAR VERTICAL	20 - 30	12
	30 - 60	15
	> 60	20
0.25:1 or 0.30:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25
0.50:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25

"RITCHIE"
Criteria for Rockfall
Catch Ditches

NOTES:

Cut heights less than 20 feet shall be treated as a normal roadway unless otherwise determined by the District Materials Engineer.

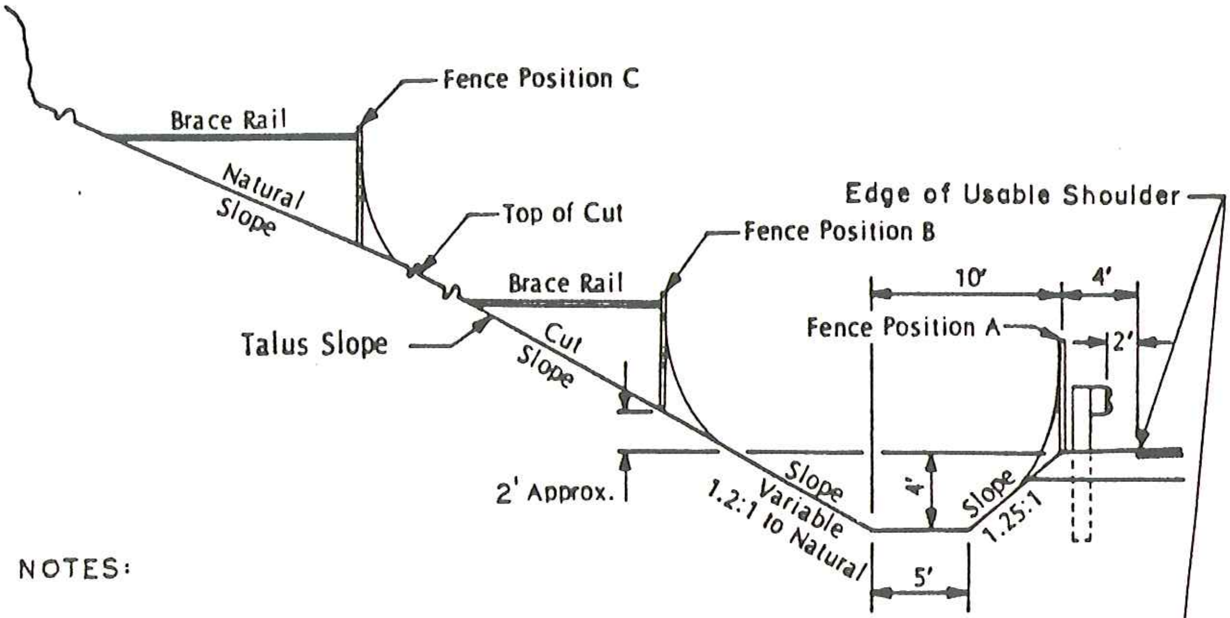
Stage 2 and 3 Alternates may be used when site conditions dictate.

Fence may be used in conjunction with the Stage 3 Alternate.

(a) Four feet when usable shoulder is 8 feet or more and centerline of barrier 1 foot from edge of usable shoulder.

(b) Two-foot minimum when usable shoulder is 8 feet or more and face of guardrail placed at edge of usable shoulder.

Figure 6-14. Roadway sections in rock cuts, design A *(Courtesy Washington State DOT Roadway Design Manual (1986).*



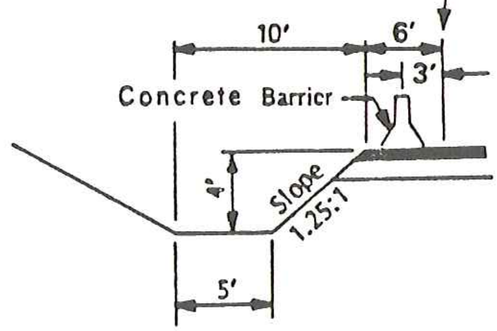
NOTES:

Ordinarily, place fence within a zone of 100 feet to 200 feet maximum from base of cliff measured along slope.

Rock protection fence may be used in conjunction with the Shoulder Barrier Alternate when site conditions dictate.

(a) Four feet when usable shoulder is 8 feet or more and centerline of barrier 1 foot from edge of usable shoulder.

(b) Two-foot minimum when usable shoulder is 8 feet or more and face of guardrail placed at edge of usable shoulder.



SHOULDER BARRIER ALTERNATE

Figure 6-15. Roadway sections in rock cuts, design B (TALUS SLOPES) *(Courtesy Washington State DOT Roadway Design Manual (1986).*

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CHAPTER 7

ROCKFALL MITIGATION METHODS

7.1. INTRODUCTION

There are three mitigation methods for prevention of rockfall and rockfall hazard.

- Stabilization concentrates on reducing the driving forces and/or increasing the resisting forces associated with the failure.

- Protection prevents rockfalls from reaching the transportation route. Protection is generally associated with shorter-term remediation of the slope and periodic maintenance or cleanup.

- Warning and instrumentation provide information that movement is occurring or an immediate warning that a failure has occurred. Warning of failure is more extensively used on railways but has specific application for highways.

These methods may be used on their own or in combination to produce cost-effective rockfall remediation or control. It must be emphasized that the designer life, maintenance requirements, and cost of rockfall mitigation is extremely variable and very site specific.

Inspection of rock slopes in areas where steep topography dominates should be completed a minimum of once a year or more frequently depending upon the factors that influence stability and severity of the hazard. Priorities need to be established to determine the locations where remedial work must be performed.

The inspection program should place initial emphasis on protection and stabilization of areas where rockfall frequency is the greatest. Locations for the remedial work should be based on the following typical factors:

- Occurrence of past rockfall;

- Obvious adverse structural geology or signs of instability;

- Obvious adverse topography above and below the highway, including a narrow ditch catchment;

- Degree of risk;

- Maintenance costs;

- Interviews with maintenance and patrol personnel;

- Costs of remedial measures and expected benefits; and

- Traffic type and frequency.

Conditions at a particular location may vary considerably. Therefore, a combination of remedial measures may need to be considered at any site.

In addition to comprehensive maintenance records, site photographs are essential in the evaluation of slope conditions before and after remedial work. A continuing record of maintenance performance is recommended. A typical summary of the types of records that should be maintained was originally developed by Piteau and Peckover (1978) and is shown in figure 7-1. Many States have now developed record keeping systems that fit their specific conditions. Statistical evaluation and comparison techniques are also used. Specific reference should be made to the FWHA Rockfall Hazard Rating System Manual (Pierson and Van Vickle, 1993).

7.2. STABILIZATION

Stabilization of a slope can be achieved by excavation of the unstable area, by reducing the driving forces that contribute to failure, or increasing the forces resisting failure. Typical procedures include the following:

- Removal of the unstable rock, particularly by scaling.

- Flatten the slope (except for block type failure).

- Reduction of the influence of pore water pressures.

- Installation of support systems.

Benches are no longer considered an applicable remedial measure for protection against rockfall. Benches gradually fill with rock and become launching pads for smaller rocks, (figure 7-2) require expensive maintenance, and invoke higher construction costs than required because of the greater excavation volumes. This is especially a concern where rock disposal sites are limited or located long distances from the site. Benches are also aesthetically undesirable and cause negative environmental impact since they result in greater over-slope height and excavation volume. Benches also have a tendency to fail over time.

7.2.1. Scaling

Scaling is an effective way to remove overhanging, protruding rocks or unstable rocks. Scaling methods are numerous and a site evaluation will determine the most cost-effective procedure for the program. Scaling at crests and on the faces of high steep slopes must be carried out by experienced personnel. The work will be performed from ropes (figure 7-3) or in a basket hanging from a boom (figure 7-4) with prybars, hydraulic splitters or jacks, and small-scale explosives or chemical expanders, such as S-Mite or Bristar.

Mechanical scaling, such as dragging a cat track across the slope with the use of a boom crane (figure 7-5), is generally a much quicker and safer method, but final hand scaling for remnant unstable rocks is still required. Mechanical methods such as the mechanical rock breaker, (figure 7-6) or the use of exploding shells from a howitzer, (figure 7-7) or high pressure water monitoring, have been used to remove unstable material. The latter allows for remote scaling. It is more commonly used for snow avalanche control.

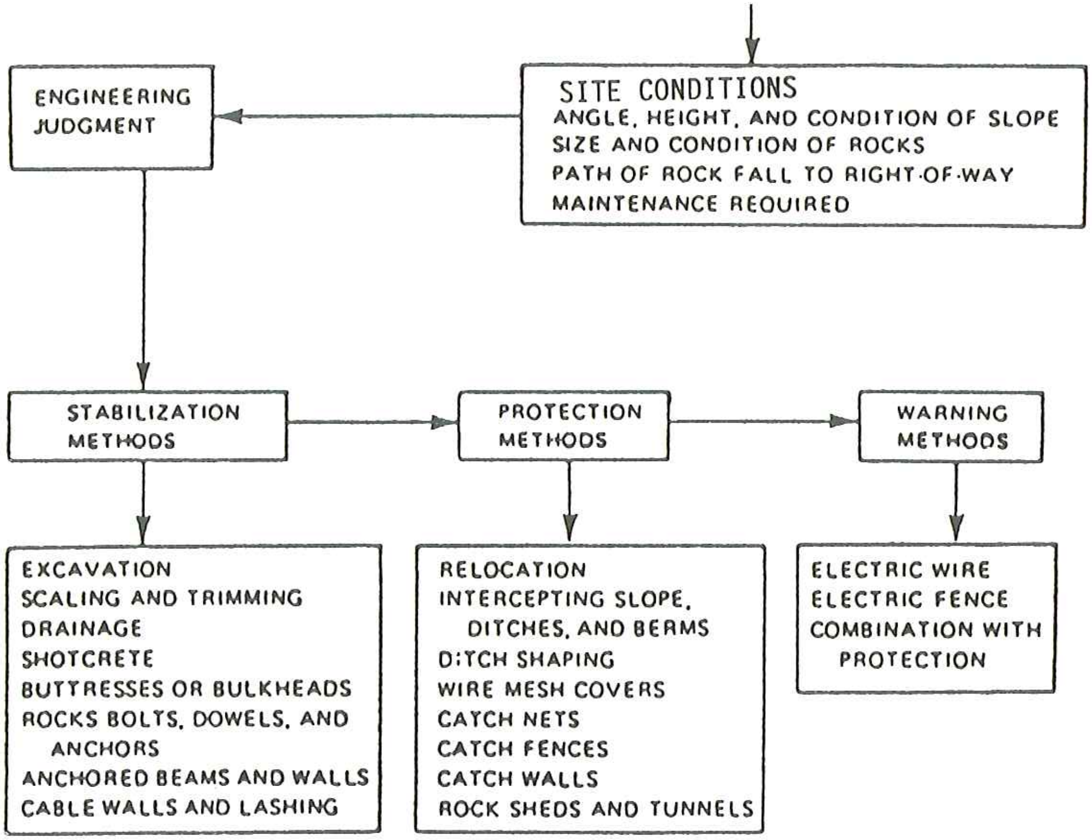


Figure 7-1. Site records and mitigation methods (Piteau and Peckover, 1978).

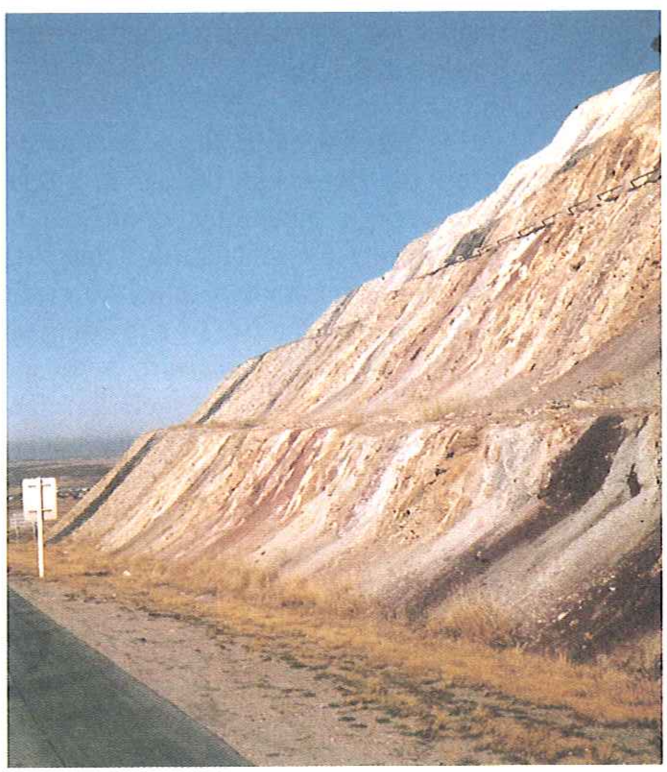


Figure 7-2. Benches developed in the slope. Experience has shown that these benches fill up with ravel and become launching pads to project rock out onto the highway. Benches also increase excavation quantities and right-of-way requirements. They are aesthetically undesirable.

Figure 7-3. Hand-scaling from ropes *(Courtesy Oregon Department of Transportation).*

Figure 7-4. Scaling from a basket mounted on a telescoping boom vehicle. The trees should be removed from the crest back for 6 feet. High winds lever rocks from the crest *(Courtesy Oregon Department of Transportation).*

Figure 7-5. Caterpillar tractor tracks being used for drag scaling to remove larger loose rocks. Final hand-scaling is required to remove smaller rocks.

Figure 7-6. Backhoe operating with a rock breaker to remove rock. This unit is used where rock is reasonably jointed and blasting could damage nearby adjacent structures. The procedure is very noisy and takes longer than blast removal *(Courtesy Learoy Excavating, Vancouver).*

Figure 7-7. Howitzer using exploding shells to scale a dangerous slope across the valley. The procedure was extremely successful. The artillerymen were contracted from the Canadian Army. Site near Hope, British Columbia. This procedure is commonly used for snow avalanches. There are times when unusual mitigation procedures are warranted.

Periodic scaling should be performed as required on some slopes. Where numerous freeze-thaw cycles occur, scaling every 8 to 10 years is desirable. In dry warm climates, scaling cycles every 12 to 15 years may be adequate. Scaling must be performed thoroughly so it is not required every several years. While it is an interim mitigation method, it is also usually the least expensive mitigation procedure. In temperate climates, such as those found in the northwest, central and eastern U.S., scaling should begin in the spring after the frost leaves the rock. Rockfall frequency is usually greater during heavy precipitation, during high winds if trees are on the slope or crest, and during spring melt.

A scaling crew will normally comprise 2 to 4 members, depending on their support method. One equipment operator, who is usually the supervisor will clear the road and load the rock. There will be a truck on call. A crane and operator also may be used to lift a basket from which scaling is performed.

A common technique to protect the roadway during scaling is to cover the road with soil and build an earth berm to control the rolling rocks.

Rockfall runnout control at the highway must be provided. Procedures for this are described later. Traffic control at the highway must be developed. Scaling will usually have to be coincide with periods of low traffic flow. Where traffic is extremely heavy, a detour may be required.

7.2.2. Removal by Blasting (Secondary Blasting)

Secondary blasting can be utilized to reduce the size of larger boulders to allow excavation and removal of debris by smaller equipment. Three common types of secondary blasting techniques are used: air cushion blasting, blockholing, and mud capping.

- Air cushion blasting provides some control over the number of fragments and the direction in which the fragments will fly. The procedure consists of drilling a blast hole 2/3 to 3/4 of the distance through the boulder. A charge of approximately 2 ounces/yd^3 (56.7g/m^3) is placed in the hole and the blast hole is stemmed with clay instead of crushed stone. The charge is placed near the center of the hole, and the hole remains empty between the charge and the bottom of the hole and the charge and the bottom of the stemming. When this technique is used, a minimum amount of flyrock will occur with the boulder popping open and laying in its original location. If more fragments are desired, the air cushion is reduced by increasing the amount of stemming.

- Blockholing is similar to air cushion blasting and requires holes drilled into the boulder, which are lightly loaded with explosive. The load is generally 2 ounces/yd^3 (56.7g/m^3) and adjusted, depending upon the rock type. If the boulder is not spherical, many small holes may have to be drilled and the powder load distributed between these holes. One hazard of blockholing is that the degree of fragmentation and flyrock can not be controlled as well as single hole air cushion blasting.

- Mud capping consists of an external charge placed on top of a boulder with a cap of mud placed on top of the charge. The mud provides a barrier to reflect a portion of the shock energy. Generally, charges of between 0.5 to 1 lb yd^3 (14.2g/m^3 to .45kg/m^3)of boulder are used. Because of the excessive noise mud capping is not recommended for use in residential areas. Details for mud cap packs can be obtained from explosive suppliers. Where no flyrock can be allowed, blasting mats should be placed over the secondary blast area (figure 7-8).

7.2.3. Chemical Expanders for Rock Breakage

There will be locations where flyrock, blast noise, blast vibration, or air or water blast forces are not acceptable. Non-explosive chemical expanders have been used to break the rock slowly during about a 30-minute to a 24-hour period. Typical materials are Bri-Star and S-Mite.

A hole or holes are drilled to 3/4 the diameter in the rock(s) to be broken. The chemical, an inorganic lime compound, is mixed with water to the supplier's specifications and poured in each drill hole to near the surface. The chemical reaction causes slow but continuous expansion until the rock breaks with negligible noise. Up to three times the number of drill holes may be required, as compared to blasting.

In order to ensure the broken rock does not roll down the slope at an uncontrolled time, the rock should be tied around with cable, which is later removed, or it should be covered with mesh, where adequate ditch catchment is not available.

Although very effective, the procedure is much more expensive than blasting.

7.2.4. Control Blasting for New Slope Excavations

Controlled blasting is an effective method to reduce damage to a rock face, to reduce long-term maintenance, and to control overbreak at the excavation limits. The concept is to limit the seismic energy created in the final rock slope. Some techniques of controlled blasting are used to provide a cosmetically appealing wall, while others are used to provide a uniform rock slope by producing a fracture plane between the holes. Long-term rock slope stability is improved when controlled blasting is used.

Figure 7-8. Blasting mats made from used tires. These cover blast areas to control flyrock from damaging adjacent facilities.

While the initial direct cost of controlled blasting will slightly exceed the cost of normal drilling and blasting, the overall benefits of reduced volume, generally steeper slopes, and reduced long-term maintenance result in a considerable long-term saving.

Control blasting techniques work best in massive rock where half of each borehole usually can be observed on the final wall. However, the absence of half casts does not indicate poor blasting practice where geologic structure exists that can impact the blasting results. Where rock has numerous joints that intersect the final wall at less than a 15° angle, it is impossible to form a smooth face. Generally, for controlled blasting to develop a uniform face with a cosmetically appealing look, joints must intersect the face at an angle greater than about 30°.

Statistical evaluation by the author of numerous rock slope inspections and seminars indicates about 85 percent of highway users find controlled blasted faces with half rounds exposed to be aesthetically pleasing.

Vertical slopes are recommended where the structural geology is favorably oriented for stability, such as near horizontal or flat dipping into the slope (figure 7-9).

Where the geologic structure dips out of the slope steeply, angled holes are generally used in controlled blasting applications. Angled drill holes along the structure assist in breakage along pre-existing joints or bedding and improves slope stability. There are advantages and disadvantages to angled drilling, some of which are listed below:

<u>Advantages</u>	<u>Disadvantages</u>
· Less backbreak	· Harder to collar holes
· Less problems at grade	· More difficult to drill
· More rock throw	· More difficult to load
· Better fragmentation	· Some holes require redrilling

There are three common types of controlled blasting; presplitting, trim (cushion) blasting, and line drilling.

A. *Presplitting*

The technique of presplitting uses lightly loaded, closely spaced drilled holes along the final slope face and fired <u>*prior*</u> to the production blast. Presplitting creates a fracture plane between holes across which the radial cracks from the production blast are reduced. Properly designed, presplitting combined with one or more buffer rows acts as a protective measure to minimize damage to the final wall from the production blast.

In most presplitting applications, the blast holes are drilled with a spacing (in feet) equal to the hole diameter (in inches). Field experimentation is recommended to adjust this ratio based on post-blast inspection. The presplit blast holes should be fired instantaneously. If line drilling (closely spaced holes along the final slope face) is used, all holes are delayed. The choice and amount of explosive charge is dependent on the rock type, strength, structural geology and slope design, and any adjacent structures. It is recommended that the explosive charge be decoupled, that is the charge diameter is smaller than the blast hole diameter. To control drift, the length of blast holes generally should not exceed 30 to 40 feet (9.2 to 12.2 meters).

Figure 7-9. Vertical rock face developed with controlled blasting. The structural geology was favorable. Any rock falling from the face will fall into the ditch. Rock quantities are minimized. The crest must be scaled and larger trees removed back at least 6 feet from the crest.

Most blast engineers prefer to load the production holes nearest the presplit line lighter than they would load the remainder of the production holes. These are normally referred to as buffer holes and are often spaced closer with smaller burdens and lighter loads than the production holes so that less energy will be directed toward the final wall.

B. *Trim (Cushion) Blasting*

Trim blasting is a control technique used to clean up a final wall after production blasting has taken place. Since the trim row along the final wall is fired *after* the production blast, it reduces the potential of blast damage to the final wall, as most of the energy is directed toward the free face. This type of controlled blasting can produce a cosmetically appealing final slope face. This procedure is generally used where the rock is weathered or heavily jointed.

Spacing of the blast holes generally is similar to the presplit technique. With cushion blasting, the burden is small. To ensure that the fractures extend between holes, the burden usually is greater than the spacing by 30 percent. Sub-drilling is minimal and stemming is generally used.

C. *Line Drilling*

Line drilling is a technique where blast holes normally are drilled within two to four diameters of one another. The holes are unloaded, or alternatively lightly loaded, closely spaced, and act as stress concentrators or guides to assist the development of the fracture between them. These unloaded holes sometimes are used in tight corners to guide the fractures into a specific angle.

For detailed recommendations on blast design refer to FHWA <u>Manual Rock Blasting and Overbreak Control</u> (1991).

7.2.5. Blast Monitoring

Vibration, air blast, and flyrock are common hazards associated with rock excavation by blasting. Vibration and airblast monitoring to measure noise and vibration are required to minimize the risk of building damage or window breakage. Several factors control these blast characteristics. Changes in burden, spacing, stemming, powder column length, number of rows, number of holes, and types of delays have an impact on vibration and airblast.

Vibration monitoring is most commonly conducted with seismic monitoring machines, such as the Instantel Seismograph. Currently, vibration levels are measured and potential damage is assessed using peak particle velocity. Particle velocity is site specific. It is generally accepted that the lower level of building damage, such as cracking of plaster, is 2 in/s (50.8mm/s). For heritage and other older structures or structures with sensitive equipment, 1.0 in/s (25.4mm/s) is recommended. Other peak particle velocity criteria, as described by the U.S. Bureau of Mines, Bulletin 656, 1971, are as follows:

- Threshold of damage (4 in/s or 101.6mm/s)
    - opening of old cracks
    - formation of new cracks
    - dislodging of loose objects

- Minor damage (5.4 in/s or 704.7mm/s)
  - fallen plaster
  - broken windows
  - fine cracks in masonry
  - no weakening of structure

- Major damage (7.6 in./s or 193mm/s)
  - large cracks in masonry
  - shifting of foundation bearing walls
  - serious weakening of structure

Ground calibration of seismic response should be performed when blasting in a new area. The two principal factors that affect vibration level are the charge weight per delay and distance. In addition, rock type, rock density, the presence and orientation of rock discontinuities, nature of terrain, blast hole conditions, presence or absence of water all combine to influence the transmission of vibration.

In the past, it has been common practice to monitor behind the blast at the nearest structure, since it was assumed this would be the location of the highest vibration levels. However, research has shown that the highest vibration levels often occur at the sides of the blast and are associated with the direction toward which the delays are progressing. In order to determine site-specific ground transmission characteristics, it is recommended that at least two seismographs be used, one placed on the end of the shot and one placed at 90° behind the shot to establish vibration levels and their relation to the measurement location.

At locations where blasting will be performed near structures, a preblast condition survey of all structures to identify, measure, and photograph cracks or previous structural damage is essential to counteract claims that the damage was caused by the blasting. Even small cracks must be identified.

Air blast is an atmospheric pressure wave transmitted from the blast outward into the surrounding area. Air blast is generated by the explosive gases being vented to the atmosphere as the rock ruptures, by stemming blow out, by displacement of the rock face or borehole, and by blasting cord initiation. Generally by limiting the peak particle velocities of the blast, the air blast will also be limited.

Blast detonation during periods of low, dense cloud cover is not recommended. The air blast may be confined and carried for extended distances.

7.2.6. Slope Drainage

Improvement of stability of a rock slope, especially if the rock mass is weak and jointed and susceptible to erosion, can improve substantially the stability of a slope. Areas behind the crest of slopes should be inspected to determine if surface water exists near the slope crest.

Surface drainage can be controlled by using the following methods:

- Drainage of water-filled depressions above the crest of the slope.
- Recontouring the slope to provide controlled surface runoff away from unstable areas.
- Concrete, slush grout, asphalt, or polypropylene can be used to temporarily or permanently seal or plug tension cracks and other highly permeable areas.
- Slush grout or asphalt lined ditches or unlined ditches in intact rock, culverts, conduits or flumes could be used to divert surface flow.
- Minimize removal of shrubbery-type vegetation cover and establish such growth.

Subsurface drainage can be established with drain holes drilled into the rock slope. Subsurface drainage will lower the water table and thus reduce the water pressure on potential failure planes. The drain holes should be designed to extend beyond the critical failure zone and intersect the maximum number of significant discontinuities. Drainage effectiveness will depend upon the geologic structure and transmissivity and orientation of discontinuities. Such drainage will also reduce the potential for ice jacking.

Heavy blasting can dislodge loose rock from adjacent slopes. To minimize the ravelling the seismic particle velocity at adjacent slopes of concern should not exceed 4 in/s (10cm/s).

Horizontal drain holes are recommended in most rock slope design. They are generally inclined upward at an angle of 3° to 5°. Perforated pipes can be used to maintain the stability of the open hole and reduce erosion (figure 7-10). To increase the effectiveness of the drainage system, high rock cuts require installation of drainage at multiple elevations on the slope. Where the rock is taken out in several lifts, drain holes should be drilled at the toe of every lift. If the rock face is stabilized with shotcrete, drain holes must be developed through the shotcrete to ensure no buildup of hydrostatic pressure.

A combination of vertical and horizontal drainage systems can be used to create a gravity feed system in a slope that consists of layered water bearing strata (figure 7-11). Sedimentary deposits can display preferential seepage paths either along contacts, between strata or within the layers themselves. The vertical drain hole (6 to 12 in/152.4 to 304.8mm diameter) is installed to draw the seepage out of the near horizontal strata. A horizontal drain hole (plus 3° to 5°) is drilled from the base of the slope to intersect the vertical drain hole and thus provide a gravity-induced system. To prevent erosion or caving, the vertical hole generally is filled with a coarse sand or gravel and the horizontal hole is lined with perforated PVC pipe. If the horizontal drain hole does not intercept the vertical drain, a small explosive charge is detonated at the estimated intersection depth to fracture the rock.

It is emphasized that where the rate of seepage exceeds the rate of evaporation adverse water pressure at shallow depth in the rock will not be recognized. The cost of the drain holes is very low, provided they are developed during construction.

Drainage can also be improved by inducing a vacuum on existing drain holes. This is established by grouting between the pipe and hole near the exit, connection of a vacuum air pump to the drain hole, and inducing below-atmospheric pressures into the discontinuities to suck out the water. This system is extremely successful on large soil and rock slides (Brawner, 1982).

7.3. SUPPORT SYSTEMS

Two types of support systems generally are used for remediation work. They are passive and active systems.

Figure 7-10. Drain holes with perforated casing installed in a rock slope. For rock cuts in excess of about 30 feet, such drains are recommended at every excavation level in the cut to reduce water pressures and freeze-thaw potential in the slope. This drain protrudes more than necessary.

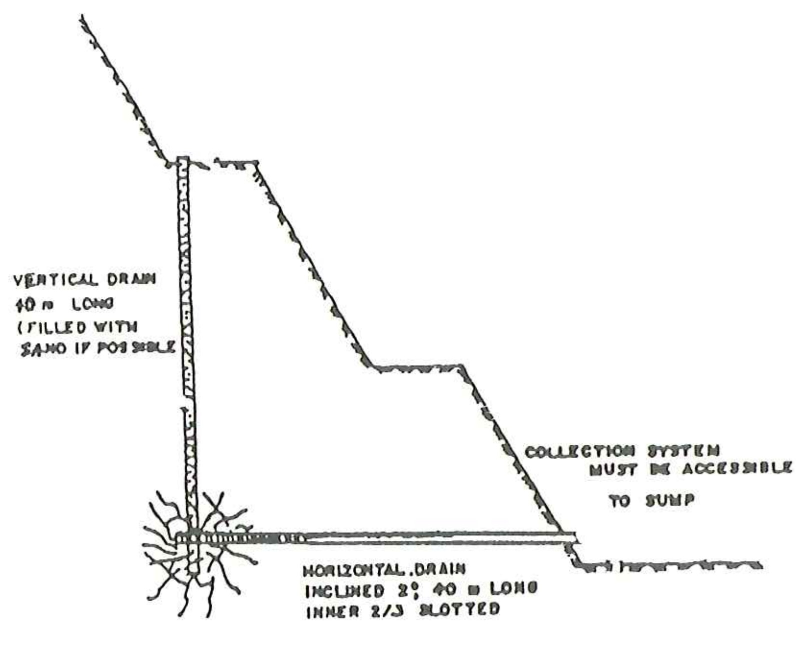


Figure 7-11. Gravity drainage system for layered strata and perched water tables. The vertical hole is 6 to 12 inch diameter and filled with fine concrete aggregate. The near horizontal drain is drilled to intercept the vertical hole.

Passive systems consist of dowels, cable lashing, shotcrete, buttresses, and retaining walls, which offer resistance to rock movement and the loads imposed by movement. Active systems consist of tensioned rock bolts and cables, anchored walls, and anchored beams, which increase the strength of the rock mass by increasing the normal load and improving the shearing resistance along discontinuities.

7.3.1. Buttresses and Retaining Walls

Structures such as buttresses are generally used to support areas where failure of an overhanging rock is possible. These structures are designed to take a portion of the unstable rock's weight, thus stabilizing the area. Buttresses are simple, effective, and permanent but are costly to construct because of the quantity of materials required. These structures generally are developed at highway level but can be as effective at higher areas of a slope.

To ensure they act as a unit, buttresses must be reinforced and anchored to the rock wall and foundation with grouted dowels. Typical examples of buttresses are shown in figures 7-12 and 7-13.

Retaining walls may consist of concrete, reinforced earth, gabions, and binwalls (figures 7-14 to 7-17). The main purposes of retaining walls are to prevent larger blocks from failing, to increase resistance against slope movement, or to develop a wider road cross section or ditch catchment. The type of wall can be selected to enhance the aesthetics of the site. Many types of walls are available.

7.3.2. Dowels

Steel dowels grouted into stable rock below potentially unstable rock will provide resistance to sliding along a throughgoing discontinuity (figures 7-18 and 7-19). At many locations, rock blocks or masses rest on discontinuities that dip out of the slope between about 25° to 60°. Multiple dowels placed along the downslope face greatly increase block stability.

Holes about 2 to 2.5 inches (50.8 to 63.5mm) in diameter and 12 inches (304.8mm) deep are drilled as close as possible to the upper block face, approximately perpendicular to the face of the lower stable rock mass.

Figure 7-12. Reinforced concrete buttress constructed to support large rock block above *(Courtesy Caltrans)*.

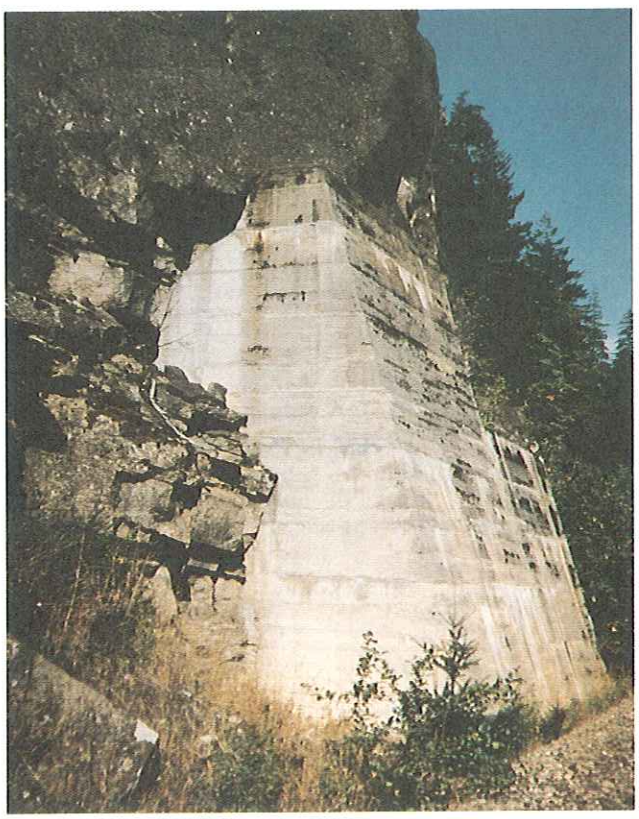


Figure 7-13. Large concrete buttress supporting very large overhanging rock. This will provide long-term stability.

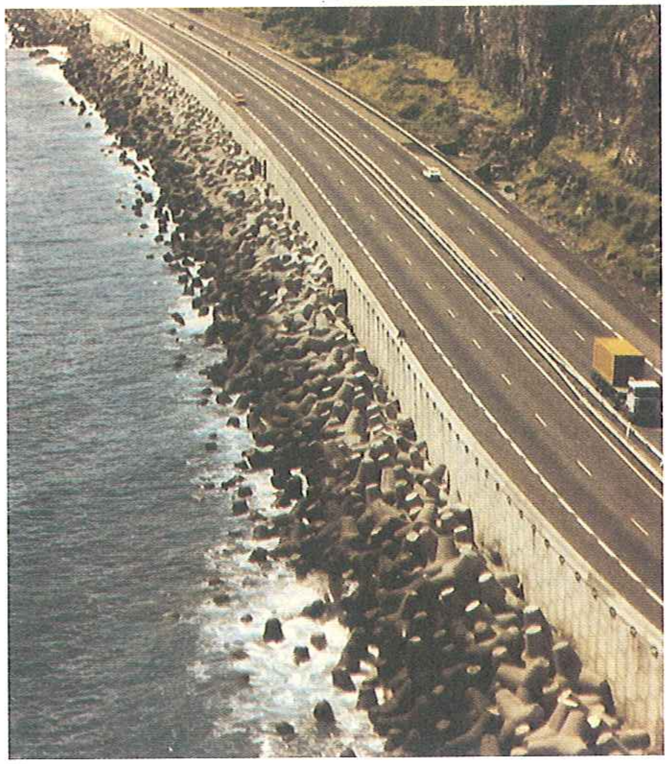


Figure 7-14. Reinforced earth wall installed to move alignment away from a ravelling rock slope. Many different aesthetically appealing facings are available with this type of construction.

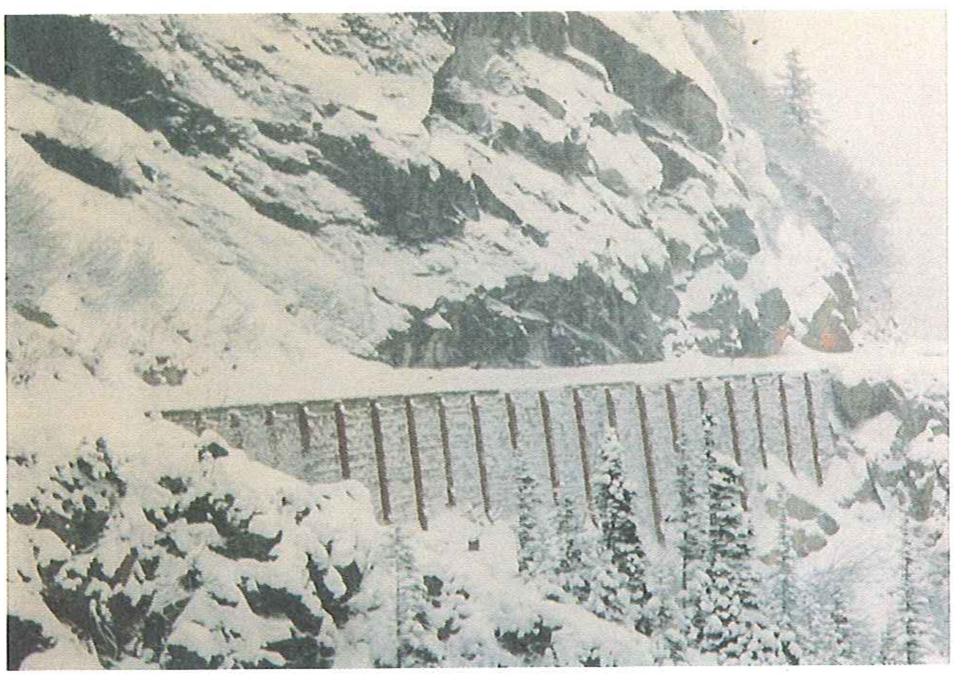


Figure 7-15. Concrete wall stabilized with tensioned anchored bolts and channel sections. This construction was used to widen the travelled section and reduce the rock excavation at the toe of a high steep bedded slope that should not be undercut.

Figure 7-16. Gabion wall placed against weathered rock to control stability. Where gabions are used near the ocean, the wires must be protected from corrosion. Gabion walls are not recommended in heavy snowfall areas where snow graders and ploughs operate, as the wires may be broken by the pushed snow.

Figure 7-17. Metal binwall placed at the inner shoulder to develop a catchment area behind *(Courtesy Caltrans)*.

Figure 7-18. Steel dowels installed at the toe of blocks that may slide on steeply dipping planes. Dry mix concrete must be packed between the dowel and the rock face so the resistance is by shear rather than bending and to prevent the rock from moving. Only minor movement will reduce the shear strength along the discontinuity.

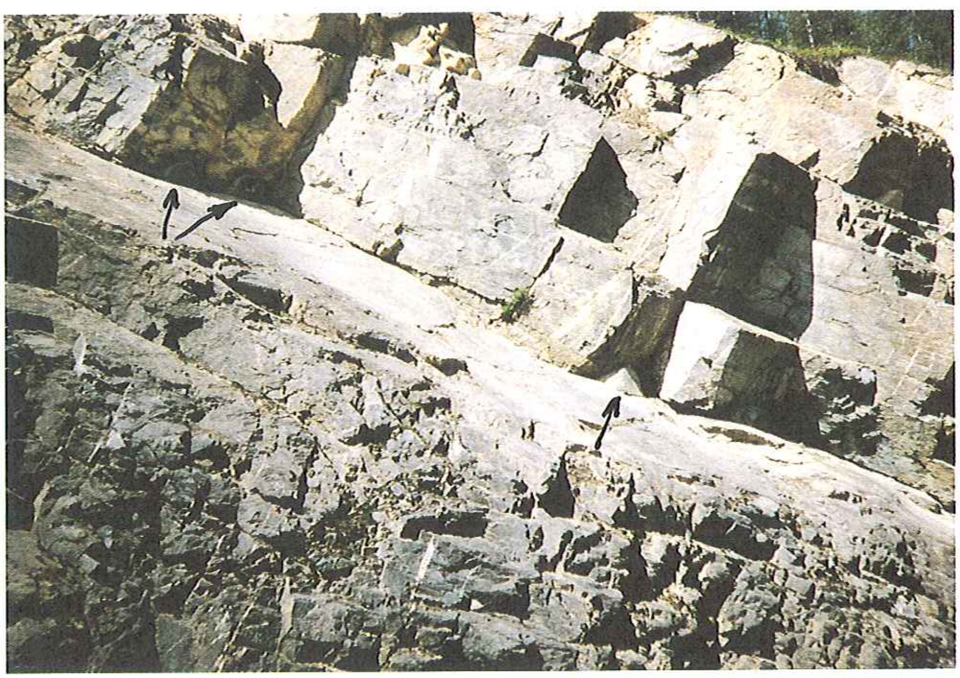


Figure 7-19. Dowels packed around with concrete to ensure they provide resistance in shear rather than bending. Each dowel will provide 20 to 30 tons of resistance.

The holes are partially filled with a neat cement or epoxy grout. Steel rebar (#10 to #11) is pushed into the hole with about 12 inches extending out of the hole. After the grout has hardened, dry mix concrete is packed between the rock block face and the dowel. If desired, the entire dowel can be embedded in the concrete so as not to be seen.

The concrete packing provides resistance from the dowel in shear rather than bending and prevents the rock from moving even a small amount which maintains peak available shear strength.

This technique is inexpensive, easy to install, and usually results in no traffic disruption. Properly installed, each dowel can resist 20 to 30 tons (18 to 27.2Mg).

Untensioned grouted steel reinforcing bars can also be installed through potentially unstable blocks into the stable rock below. These installations are referred to as passive dowels. The concept is that if the upper rock moves the bolt will go into tension and increase the normal load on the potential failure zone. The angle of installation must be selected such that the bar will not bend and the block will not start to override the surface roughness. It is recommended that any passive dowels should be installed at 10 to 15° below the perpendicular to the lower block. This will ensure the bar goes into tension as soon as the smallest movement occurs and maximum steel resistance is provided by shear.

Passive dowels should be designed only to provide about 50 percent of the stabilizing strength of comparable tensioned grouted anchor bolts since they do not increase the normal load of the block that develops frictional resistance along the joint. The latter are preferred.

Corrosion resistant dowels should be used. The dowels should be grouted full length.

Dowels can be used to support small to very large rock blocks.

7.3.3. Cable Lashing

Cable lashing should be considered where a rock mass is potentially unstable and for reasons of safety to traffic, difficulty of access, or danger of removal, and it is better to hold the rock in place. Drillers who express concern that the vibration of their drills may cause the rock to fail should have their concerns heeded. Typical examples of cable lashing are shown in figures 7-20 and 7-21.

Short-term methods of stabilization consist of anchored cable nets or cable lashing. Anchored cable nets can be used to stabilize loose blocks or larger rocks up to 5 to 8 feet (1.5 to 2.4 meters) in diameter. This remedial support acts like a sling and extends around the surface of the unstable mass. The cable net is gathered on each side by main cables leading to rock anchors.

Cable lashing involves tying or wrapping unstable rocks with individual cable strands anchored to the slope. This technique is a simple, economical restraint for larger rocks.

The volume and weight of the rock should be estimated. Eye bolts should be grouted into competent rock on either side of the unstable rock. The eye bolts and grout bond should be designed to carry the entire weight of the rock with a safety factor, plus an extra 20 percent in earthquake areas. Sufficient eye bolts and high-tensile steel cable should be lashed to hold the rock in place. Turnbuckles should be used to tension the cable to a predetermined torque. All components should be corrosion resistant or corrosion protected. The turnbuckles should be retightened about one month after installation and checked annually thereafter. Tensioning should be done during warm weather.

7.3.4. Shotcrete

Shotcrete is utilized frequently when treating unstable rock slopes and is used primarily to prevent weathering and spalling of a rock surface, as well as *knit* together the surface of a slope (figures 7-22 and 7-23).

Shotcrete consists of mortar and aggregate projected onto a rock surface by an air jet. Generally, for rock slope stabilization, the material is applied in one 2-inch to 3-inch (50.8 to 76.2mm) layer.

With the addition of steel fibers to the mix, recent developments in shotcrete technology have provided superior strength and durability. These steel fibers replace

Figure 7-20. Cable lashing to hold blocky rock in place. It was considered too risky to try and rock bolt or remove these rocks prior to lashing. Four tensioned grouted rock bolts were installed after cable lashing.

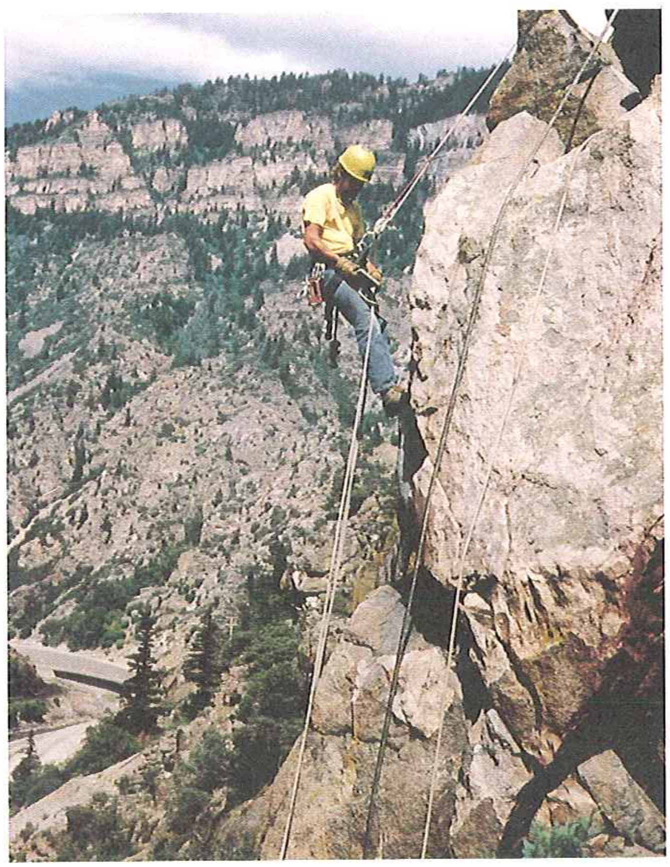


Figure 7-21. Cable lashing in progress. The turnbuckles should be retightened after about one month and annually thereafter *(Courtesy Colorado Department of Transportation).*

Figure 7-22. The overburden bouldery glacial till above bedrock has been shotcreted to maintain stability. The shotcrete has been tied to the slope with grouted steel anchors. Drain holes were located near the base of the shotcrete.

Figure 7-23. Shotcrete installed in a blocky rock face in 1958. The face has weathered to a natural rock color. This example indicates the long-term stability of well-placed shotcrete. Today's applications using steel fiber reinforcement and Silica Fume admixture further improve longevity.

most applications where welded wire fabric was previously utilized. The fibers increase the tensile strength of the shotcrete by providing numerous bonding surfaces within a small area. The fiber reinforcement also reduces the risk that shrinkage cracks will develop during curing. Many States now use steel fibers to replace wire mesh. The cost of shotcreting is reduced since only one step is required, and thickness and rebound are reduced. However, wear on the shotcrete equipment is increased. Where lower tensile strength is required, epoxy fibers have been used.

Welded wire fabric is still recommended to reinforce shotcrete applications on weathered rock, soil, or across faults. The mesh is tied to grouted steel anchors and shotcrete is then applied (figure 7-24). A disadvantage of using a welded wire fabric is the problem of moulding the fabric to the slope contours. Where surface contours are very irregular and large gaps develop between the mesh and surface, the shotcrete will tend to bond poorly to the rock face. This location may spall or weather more quickly and cause a future maintenance problem. To stabilize weathered surfaces, the mesh is tied to grouted steel anchors and then shotcrete is applied (figure 7-24).

Several admixtures have been utilized to improve the characteristics of the shotcrete. A recent development is the use of Silica Fume, a byproduct of the Ferro Silicon industry, as an admixture that provides up to a two-fold increase in compressive strength, as well as increased viscosity. The added viscosity allows a greater thickness to be applied to a surface without slumping of the material or where conditions are wet.

Shotcrete is applied by either wet or dry application. The wet mix involves mixing the shotcrete to specifications at a central plant then transporting it to the site. For a dry mix, additives and mortar are mixed on site and pumped via compressed air to the nozzle of the assembly at which point the nozzleman controls the final amount of water added into the mix. Gaging pins are used to ensure the proper thickness of shotcrete is in place.

Slope drainage is essential when shotcrete is applied. Drain holes should be installed to reduce water pressures behind the shotcrete, as well as improve the long-term stability. Rather than drill holes through the shotcrete

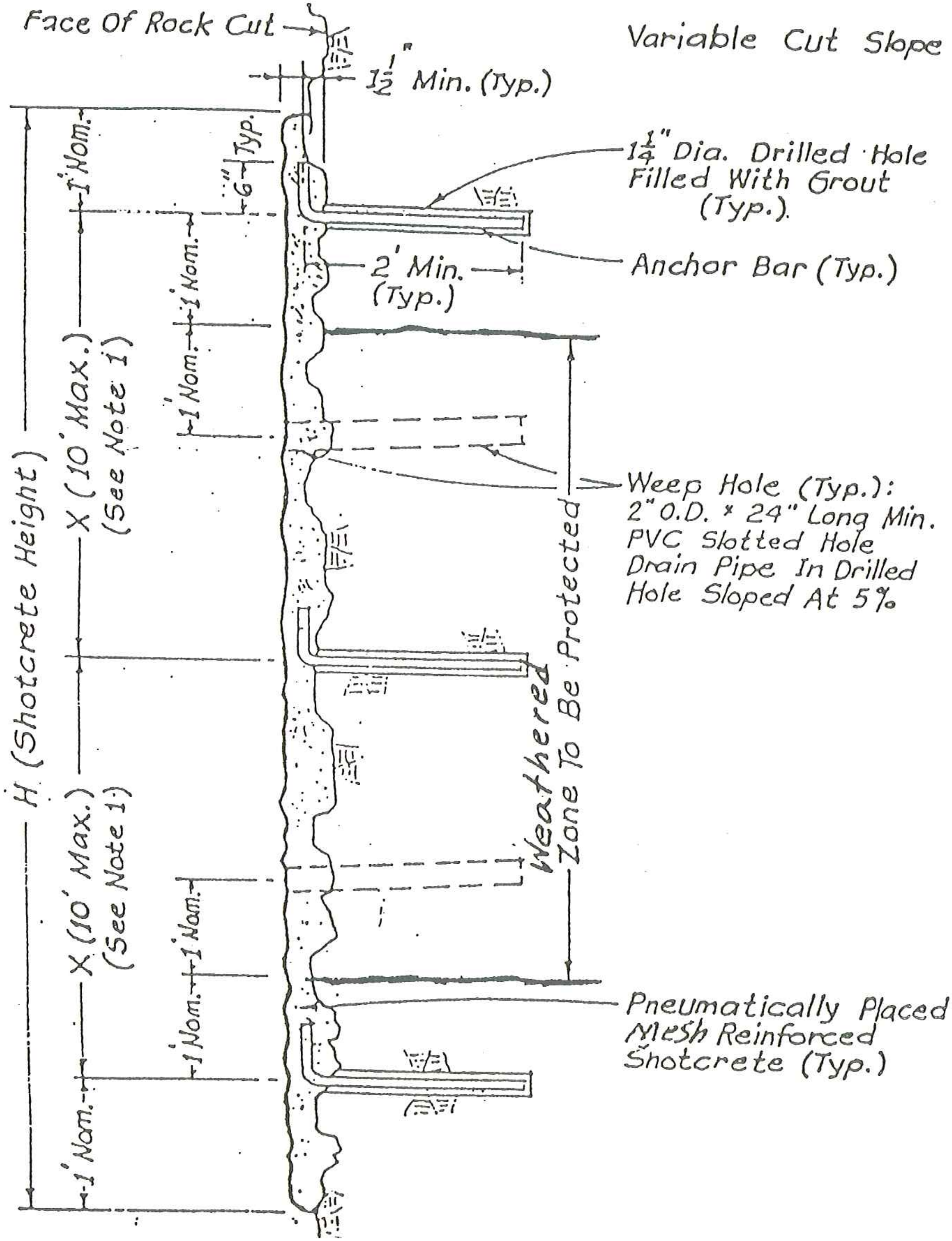


Figure 7-24. Mesh reinforced shotcrete anchored to rock to protect weathered or faulted zone *(Courtesy Oregon Department of Transportation)*.

(which may not intersect discontinuities), it is recommended that survey stakes be driven into joints or where seeps exist and shotcreted around. After the initial set of the shotcrete, the survey stake is removed to provide the drain hole. Prior to shotcrete application, the rock surfaces must be cleaned of road oil, dirt, moss, and vegetation by air or water jet and wetted to ensure good bonding.

The most important advantage of shotcrete is that it offers the engineer a rapid, mechanized, and uncomplicated solution to stabilizing blocky rock slope faces. The rock face should be scaled before shotcreting. Where shotcrete is applied during hot weather, curing compounds may be required.

Specifications for shotcrete application must emphasize the experience of the nozzleman, thickness control, rebound control, and application technique. Concrete thickness should be controlled with gaging pins on about 5 foot (1.5m) centers.

Coloring can be added to the shotcrete for aesthetic purposes. This can be done either by the addition of a coloring agent to the shotcrete mix or by staining after application on the rock face. However, the shotcrete naturally weathers over a number of years.

7.3.5. Rock Bolts

Rock bolts reinforce a rock mass and increase the shear strength along the discontinuity so that the stability of the block is increased (figure 7-25). Rock bolts are generally used to stabilize surface or near surface rock while anchor cables are used to support large unstable rock masses. The latter are beyond the scope of this manual. The system is active in that the bolt exerts a compressive force on the rock, thus preventing relaxation, frost heaving, or elastic rebound of the rock mass. The increase in the normal load across unfavorably oriented discontinuities increases the shear strength along the discontinuity. Generally, the anchorage length should be about one-third to one-half the depth to the failure surface.

The primary advantage of an active or prestressed bolt over a passive type system is that no movement has to take place before the prestressed anchor develops its full

Figure 7-25. Rock bolts installed to stabilize a foliated granite slab. It is preferable to tension and grout the bolts to increase the shear strength and reduce the corrosion potential.

capacity. Thus, tension cracks and deformation of the slope is minimized and the peak strength along the discontinuity is retained.

The passive type of steel reinforcement consists of untensioned, grouted steel bars called dowels and are generally used only to increase the shearing resistance across a potential failure plane. Used in combination with other support methods such as steel strapping, wire mesh or shotcrete, and buttresses, dowels improve stability of the slope.

Both active and passive types of reinforcement require that a sufficient length of the bolt be anchored beyond the possible failure plane. Cement grouted, epoxy resin, or mechanic anchors can be used to anchor the rock bolt to the intact rock in the borehole. The Williams Hollow core expansion shell rock bolt is shown in figure 7-26. It is more commonly used in hard rock where the mechanical anchor works well. Grout provides corrosion protection for the bolt and locks in the tension applied. Grouting is essential when long-term stability of the slope is required.

Dywidag threadbar resin anchored rock bolts (figure 7-27) commonly are used for rock stabilization. They have an advantage in that they can be cut to specific lengths and have application in weaker rock where mechanical anchors may fail.

When polyester grouts are used, a rapid time set polyester is normally employed to develop the anchorage, and a longer time epoxy is used for the remainder of the bolt after tension has been applied.

Tensioned grouted bolts are preferred to grouted dowels because only half the number are required to develop the same stability.

Rock bolts and chain link mesh (figure 7-28) provide an effective method of stabilizing ravelling rock slopes. Rock bolts and steel strapping (figure 7-29) have been used to stabilize badly weathered rock slopes. The mesh and strapping primarily prevent rock falls and small blocks from dislodging on the slope, while the rock bolts pin the mesh to the face as well as provide deeper stabilizing forces to knit the mass together and prevent large rock mass failure.

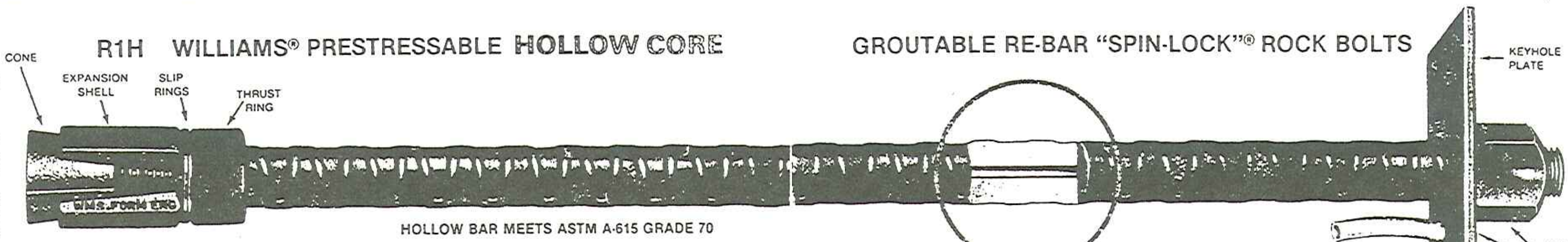
Where short-term stabilization or emergency stabilization is required, the Split set friction rock stabilizer (figure 7-30) or Swellex rock reinforcement system (figure 7-31) are effective, rapidly installed, and inexpensive.

The Split Set rock stabilizer is simple. Each is a long tube of high-strength steel, with a slot along its entire length. One end is tapered for easy insertion into a drilled hole in the rock. The other end has a welded ring flange to retain a base plate.

The Split Set stabilizer is driven into a hole roughly two inches longer and slightly smaller in diameter than the tube. Because of the slot along its length, the entire stabilizer is compressed. This enables it to exert a powerful outward force against the rock, anchoring itself tightly and securely in the rock.

"SPIN-LOCK" HOLLOW CORE ROCKBOLT "SPIN-LOCK"® HOLLOW CORE ROCKBOLT

R1H WILLIAMS® PRESTRESSABLE HOLLOW CORE GROUTABLE RE-BAR "SPIN-LOCK"® ROCK BOLTS



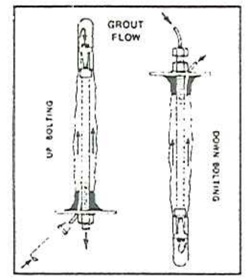
HOLLOW BAR MEETS ASTM A-615 GRADE 70

PRESTRESSABLE • POSITIVE GROUTING • PERMANENT

Through years of development, Williams has produced and patented the only Prestressable, Hollow-Core Groutable, Rebar, Spin-lock Rock Bolt System. The "hollow-core" allows the bolt to always be grouted from the lowest gravitational point. In an up bolting situation, the grout is pumped in through the plastic grout tube and begins to fill the drill hole from the plate. The grout rises until the entire hole is filled and the grout returns through the hollow bar. In down grouting situations, the grout is pumped through the hollow bar and starts at the bottom of the hole. It rises and returns through the de-air tube when the hole is filled.

This grouting procedure is a critical point in creating a permanent rock bolt, since an ungrouted bolt is subject to rock relaxation and deterioration. In the Williams hollow-core system, as the grout rises it spreads through the many fissures and voids to solidify the rock and create monolithic section including rock, grout and bolt. The bolt is completely engulfed in grout and is protected from corrosion. This is extremely important and only obtained with Williams hollow-core grouting systems since epoxy and pre-measured cement grouting systems do not allow for the grout which spreads through the rock. The deformed, high-bond rod of the US-HC series, is manufactured by a special process from a high grade of steel substantially exceeding the strength of common re-bar stock. The "SPIN-LOCK®" head assemblies provide 300° perimeter expansion anchorage and are designed to develop the full strength of the hollow-core rod.

Because the "SPIN-LOCK"® head assembly develops the full strength of the rod, the Hollow-Core rock bolt may be prestressed to the desired design load and tested prior to grouting.



ADVANTAGES OF WILLIAMS® HOLLOW CORE over two tube or resin systems

RESIN OR PRE-MEASURED GROUT SYSTEMS

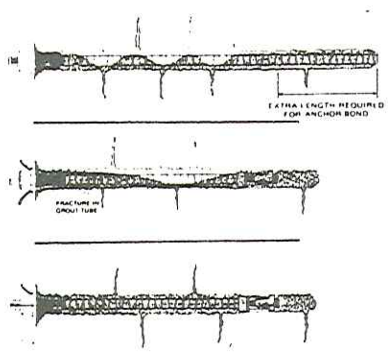
1. Fissures, voids, or rock fractures cannot be pre-measured to determine the proper amount of grout required.
2. Water or other debris is left in the hole and is mixed with grout.
3. Bolt is poorly grouted and left exposed to corrosion and failure.

TWO TUBE SYSTEM

1. Grouting tube is very susceptible to damage during installation.
2. Damage to the tube may result in premature grout return.
3. Poorly grouted bolt is left susceptible to corrosion and failure.

WILLIAMS® HOLLOW CORE SYSTEM

1. Grout is pumped through the bolt forcing out water and debris and replacing with grout.
2. All fissures, voids, and fractures are pressure grouted starting at the lowest gravitational point. Visable return of grout completes inspection.
3. Bolt is permanent. Safe from corrosion or failure.



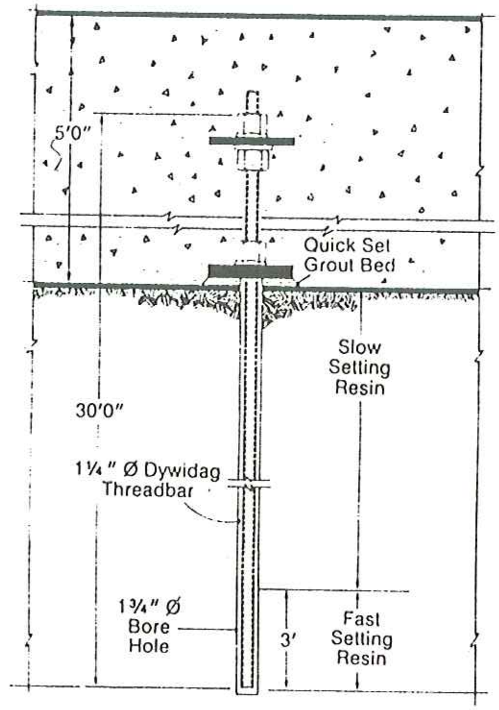
WILLIAMS® HOLLOW CORE REBAR ROCK BOLTS

DIA.	RECOMMENDED DESIGN LOAD AT APPROX. 2:1 SAFETY FACTOR	MAXIMUM WORKING LOAD TO ELASTIC LIMIT	ULTIMATE STRENGTH	ROCK TYPE	DRILL HOLE DIA. (1)	TYPE HEAD ASS'Y	TORQUE FT./LBS. TO EXPAND SHELL (2)	TORQUE FT./LBS. ON NUT	PART NUMBER	REFERENCE NUMBER
1" - 8 / 25 mm	25,000 lbs / 11,350 kg.	37,000 lbs / 16,800 kg.	50,000 lbs / 22,700 kg.	HARD & MEDIUM	1-5/8" - 41 mm	A 13	200	300	R1H-08-A 13	US-8-HC-SCS-158
				HARD & MEDIUM	1-3/4 - 44 mm	A 14	250	300	R1H-08-A 14	US-8-HC-SCS-175
				HARD & MEDIUM	1-7/8" - 48 mm	A 15	250	300	R1H-08-A 15	US-8-HC-SCS-178
				MEDIUM & WEAK	1-5/8 - 41 mm	B 13	250	300	R1H-08-B 13	US-8-HC-LCS-158
				MEDIUM & WEAK	1-3/4" - 44 mm	B 14	250	300	R1H-08-B 14	US-8-HC-LCS-175
				MEDIUM & WEAK	1-7/8 - 48 mm	B 15	250	300	R1H-08-B 15	US-8-HC-LCS-178
				WEAK ROCK & CONCRETE	1-3/4" - 44 mm	C 14	250	300	R1H-08-C 14	US-8-HC-LCSF-175
				WEAK ROCK & CONCRETE	1-3/4" - 44 mm	D 14	250	300	R1H-08-D 14	US-8-HC-LCSLF-175
1-3/8" - 8 / 35 mm	50,000 lbs. / 22,675 kg.	74,000 lbs / 33,550 kg	100,000 lbs. / 45,350 kg.	HARD & MEDIUM	2" - 51 mm	B 16	250	NOTE (3)	R1H-11-B 16	US-11-HC-LCS-200
				MEDIUM & WEAK	2-1/4 - 57 mm	B 18	550	NOTE (3)	R1H-11-B 18	US-11-HC-LCS-225
				MEDIUM & WEAK	2-1/2" - 63 mm	B 20	550	NOTE (3)	R1H-11-B 20	US-11-HC-LCS-250
				WEAK ROCK & CONCRETE	2-1/4" - 57 mm	C 18	750	NOTE (3)	R1H-11-C 18	US-11-HC-LCSF-225
				WEAK ROCK & CONCRETE	2-1/2" - 63 mm	C 20	750	NOTE (3)	R1H-11-C 20	US-11-HC-LCSF-250
				WEAK ROCK & CONCRETE	2-1/4" - 57 mm	D 18	750	NOTE (3)	R1H-11-D 18	US-11-HC-LCSLF-225
				WEAK ROCK & CONCRETE	2-1/2" - 63 mm	D 20	750	NOTE (3)	R1H-11-D 20	US-11-HC-LCSLF-250
2" - 6 / 51 mm	100,000 lbs. / 45,350 kg	148,000 lbs / 67,150 kg	200,000 lbs. / 90,700 kg.	ROCK & CONCRETE	3" - 76 mm	D 24	1000	NOTE (3)	R1H-16-E 24	US-16-HC-LCSHF-300
				WEAK ROCK & CONCRETE	3-1/2" - 89 mm	D 28	1000	NOTE (3)	R1H-16-D 28	US-16-HC-LCSLF-350

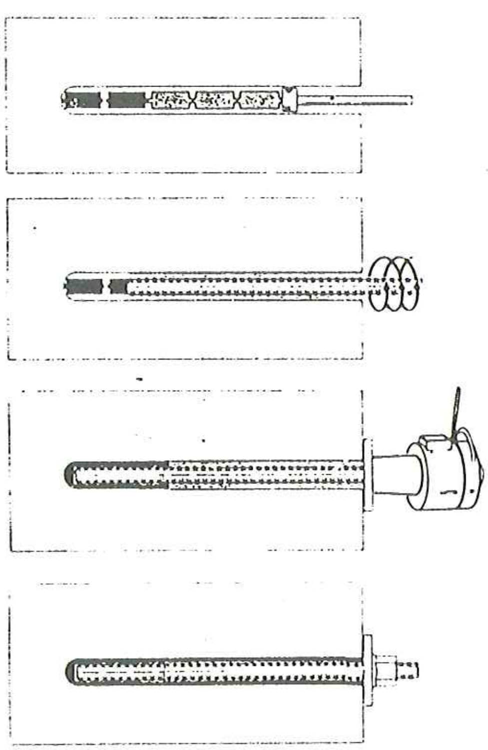
NOTES:

(1) Care should be taken to prevent drilling oversized holes.
(2) A function of strata strength. More torque may be required on long bolts or in special rock conditions. Consult your Williams Representative for more specific details.
(3) Stress to desired tensile load using a hollow ram hydraulic jack. Consult your Williams Representative.
(4) WILLIAMS reserves the right to ship full length or coupled units as necessary.

Figure 7-26. Williams Hollow Core expansion shell rebar rock bolt. After the expansion shell is anchored, the bolt is tensioned to the desired stress and then grouted. The grouting locks the stress in and minimizes corrosion *(Courtesy Williams Ltd.)*.



Stilling Basin Rock Anchors, Smithland Dam, Ohio River, U.S. Army Corps of Engineers, Nashville District. 1¼" Ø Grade 150 Dywidag Threadbars.



Drill and clean hole.
Drill smallest diameter hole which is compatible with the bolt and cartridge diameter selected.

Insert Resin Cartridges.
Use fast setting resin for bond length. if full encapsulation is desired, use slow setting resin or cement grout in the upper length to accommodate stressing

Insert DYWIDAG Threadbar.
Spin bar with drill tool at about 100 rpm. Advance Threadbar through cartridges while spinning. Spin for 30 to 60 seconds after reaching the bottom of the hole. Total spinning time should not exceed gel time.

Mount bearing plate and secure plate with anchor nut.
Wedge washers are required when anchor plate is not perpendicular to bolt.

Stress after resin has cured (when required)
Setting time varies from 1 to 20 minutes depending upon temperature and the resin type used. Apply tension with hydraulic jack, torque wrench or air wrench.

Monitor bolt tension.
The bolt tension is monitored by reading the pressure gauge where the hydraulic jack is utilized Where torque or air wrench is utilized, the bolt tension is monitored by developing a tension-torque relationship curve for the specific application.

Figure 7-27. DYWIDAG Resin Anchor Installation

Figure 7-28. Blocky rock slope stabilized with rock bolts and chain link mesh. The mesh is intended to hold the rock in place. This procedure is used more frequently at tunnel portals and where rock conditions change over a short distance.

Figure 7-29. Blocky weathered rock slope stabilized with rock bolts and steel strapping. To improve appearance the strapping can be plastic color coated.

Figure 7-30. Split set friction rock stabilizer used for temporary, short term and emergency stabilization. Maximum length is 8 feet (*Courtesy Ingersol Rand*).

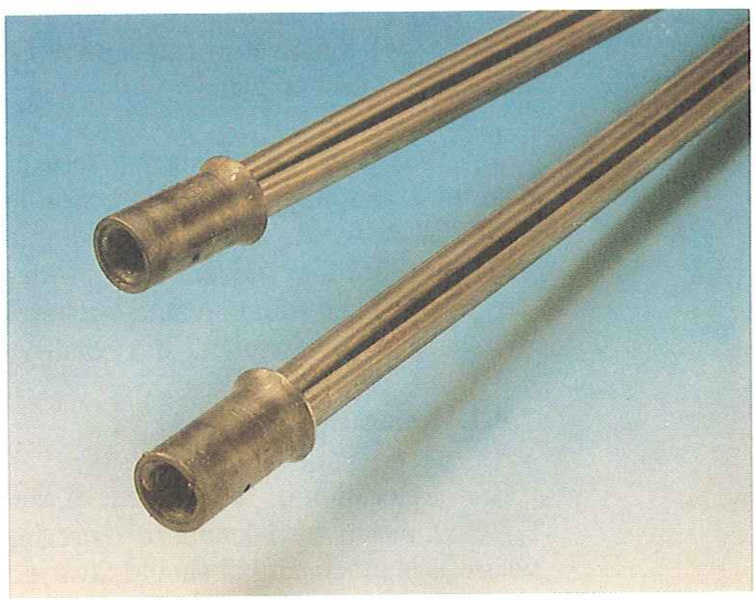


Figure 7-31. Swellex rock reinforcement system. Used for temporary, short term or emergency stabilization. Sections can be added together to develop bolts 20 to 30 feet long *(Courtesy Atlas-Copco)*.

The Swellex bolt is a steel tube with an outer diameter of 1.6 inches (40.6mm) and a wall thickness of 0.8 inches (20.3mm). The tube has been reshaped to an outer diameter of only 1 inch (25.4mm). The bolt is installed into a drilled hole. High pressure water is injected into the bolt, expanding it to fill the hole and conform to irregularities in the hole. Several sections of bolt can be connected together to make longer bolts.

These bolts are very thin walled and subject to corrosion. Hence they are not recommended in corrosive environments. Research is ongoing to increase the corrosion resistance and life expectancy.

7.4. PROTECTION METHODS

Protection methods differ from the rock stabilization techniques previously discussed in that these methods do not prevent rock falls but will prevent from them reaching the roadway. Usually a barrier or designed catchment area is employed. Prior to the design of a slope protection method, it is important that the characteristics of the rock falls and their frequency be evaluated (See hapter 6). In some instances, realignment or relocation of the highway may be feasible and economical.

The path that a bouncing rock takes is difficult to determine and may require protection methods such as a high wall or fence. Rolling or sliding rocks are easier to intercept since they are in contact with the slope. Properly designed ditches with or without a barrier on the inner shoulder are successful for this condition.

7.4.1. Relocation

Relocation or realignment of the highway will be an effective means of reducing or removing the rockfall hazard where it is practical and should always be considered. Unfortunately, it may also be the most expensive solution. A decision to relocate or realign the highway must take into account costs of construction and impact of any traffic delays and weigh them against the savings in future maintenance costs, remedial work, and litigation costs on the existing route. Relocation is possible when space is available and the alignment and design can be improved. In areas of severe rockfall conditions, tunnels and rock sheds may be considered.

7.4.2. Slope Remediation-Benches

For years, the slope design of many high rock slopes in North America have used intermediate slopes and benches. The benches were initially considered to intercept rock that falls before it reaches the highway level. The original premise was that the benches would be cleaned periodically. However, experience has shown that these intermediate benches are usually not cleaned and that they act as launching ramps for rock falls, projecting some of them into traffic areas. In addition to minimal protection provided by these benches, they also create a maintenance problem and require periodic cleaning and removal of accumulated debris. With the improvement of blasting techniques in the last decade, rock slopes can be safely excavated to considerable height without intermediate benches and at steep angles with little slope damage. The presence of a well-designed catchment at the toe of the slope eliminates the need for benches except at the contact of soil overburden and the rock. The saving in rock excavation is usually substantial. More detail is presented in chapter 8.

7.4.3. Draped Mesh

An effective method of slope protection is the use of draped mesh over the slope. Wire mesh is a versatile and economical method of prevention of rockfall from reaching the highway. The mesh, preferably a Gabion type, wire mesh, or chain link, is draped from anchored cables at the top of the slope over the face (figures 7-32 and 7-33). The mesh may or may not be anchored to the face. Anchoring the mesh holds the rock in place and reduces rock removal in the ditch. The mesh must be strong enough to hold any loose rock. Leaving the mesh loosely draped allows the rock to ravel down inside the mesh into the ditch.

Wire mesh is suitable where the rock mass is well-fractured, the rocks are not larger than about 2 feet (6.1 meters) and the slope possesses a reasonably uniform face with limited protrusions. Where heavy snowfall or ice glaciers develop on the slope, the weight may tear the mesh.

Figure 7-32. Wire mesh draped from the top of the rock slope. The mesh is draped from cables attached to trees or eye bolts grouted into the rock. Loose rock ravels down inside the mesh and drops into the ditch. Periodic excavation of the rockfalls from the ditch is required.

Figure 7-33. Draped wire mesh on the Olympic Peninsula Highway south of Port Townsend, Washington installed about 1960.

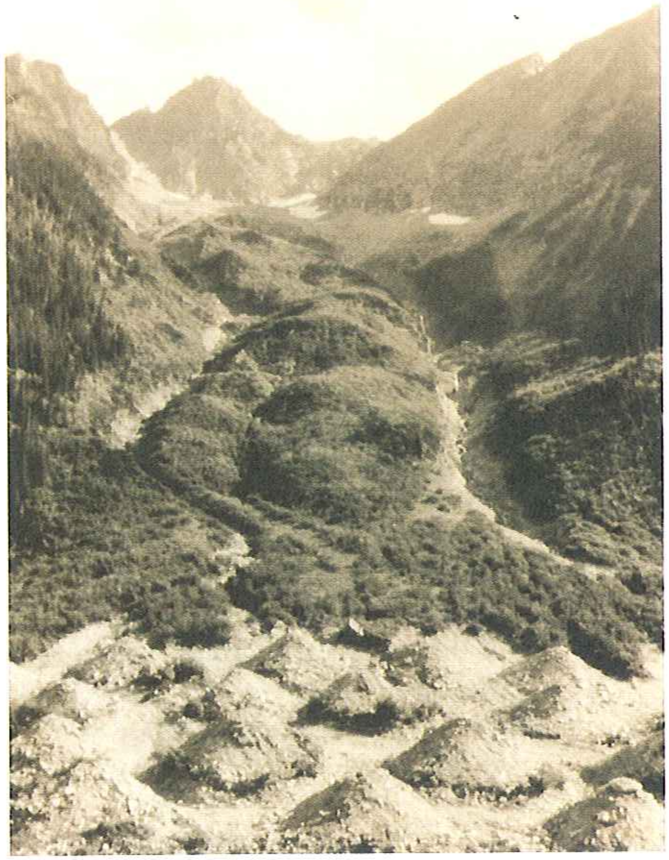


Figure 7-34. Energy dissipating mounds constructed to stop rolling rock by forcing them to change direction and loose energy. These will develop growth coverage in a short time.

Where the size of the rockfall that may ravel is large (approx. 1 yd^3 or .765m^3), stronger mesh is required. For this condition, Brugg type cable mesh, woven wire-rope mesh, or submarine netting should be considered.

7.4.4. Diversion Berms or Mounds

Where a defined channel of ravelling rock occurs and a flatter slope exists above the highway, it may be possible to develop a diversion berm to change the direction of rolling, bouncing rock away from the highway. Alternatively, energy dissipating mounds (figure 7-34) more frequently used for avalanche control can reduce rolling rock that reaches the highway. Each time the rock is forced to change direction, it loses some energy.

7.4.5. Ditch Treatments

Properly designed inner catch ditches will intercept much of the rockfall from a talus run or rock slope. The ditch geometry (depth, width, shoulder slope, shoulder fence or jersey barrier) has a great influence on the success of catch ditches. Details of rockfall, trajectory analysis and ditch cross sections are presented in chapter 6.

Figure 7-35 illustrates a totally inadequate catchment ditch. Figure 7-36 shows a ditch with adequate width but inadequate depth considering the face slope angle. Figure 7-37 shows a very wide ditch with a small gauge wire catch fence.

Where the structural geology is favorable, vertical rock slopes can be used and the catch ditches can be narrower. In any event, the ditches should be wide enough to allow maintenance equipment into the ditch to clean out the rockfall without having to work off the highway.

Many States are now using a modification of the Ritchie design concept developed in 1963. One ditch design of the Washington State Department of Transportation is shown in figure 7-38. A flat-bottom ditch is recommended in moderate to severe rockfall conditions. This design may include catch fences.

Where heavy snow-fall occurs a catch ditch may require development in the snow to provide catchment area during the winter and spring melt period.

7.4.6. Catch Fences

The States of Washington and Oregon have developed catch fences to be used at the toe of the slope or on the shoulder in conjuction with the modified Ritchie ditch design. Each design is site specific.

The mesh is galvanized steel gabion wire mesh fabric with a nominal diameter of 0.12 inch and a minimum tensile strength of 60,000 psi. Maximum mesh size should be approximatley 4.75 inces with triple twist and hexagonal shape.

The mesh is tied by hog rings to 3/8 inch wire rope with a minimum breaking strength of 13,000 Ibs.

Line posts are generally 4 inch. O.D. pipe size galvanized steel pipe. Anchor spring assemblies are installed at each end of fence. Typical fence detail is shown in figure 7-39.

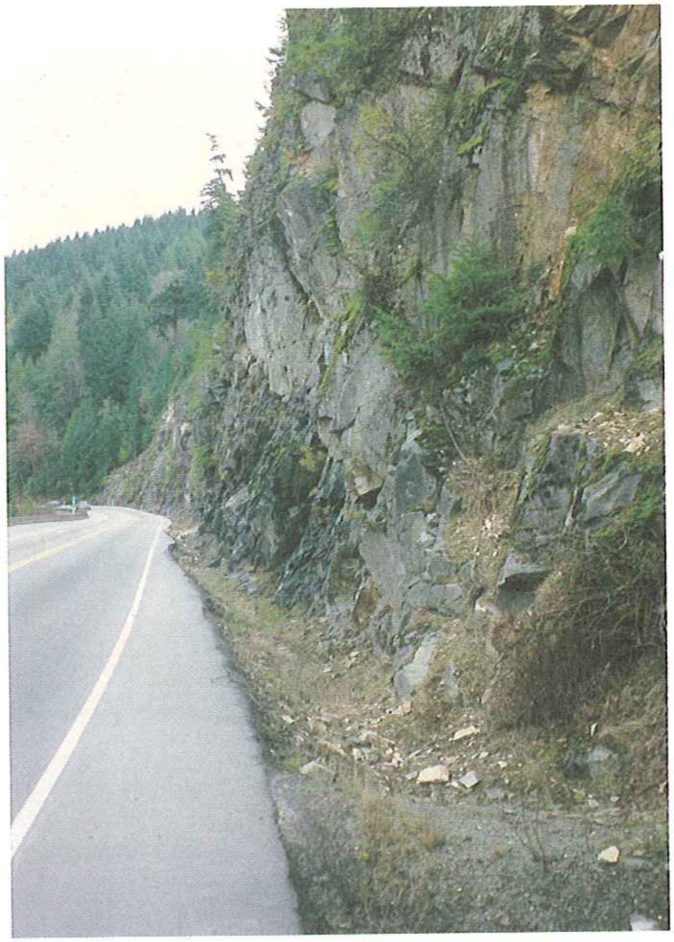


Figure 7-35. A narrow ditch that has very limited capacity to catch rock. This design is totally inadequate today.

Figure 7-36. A ditch that is adequate width, but has inadequate depth and inadequate road shoulder.

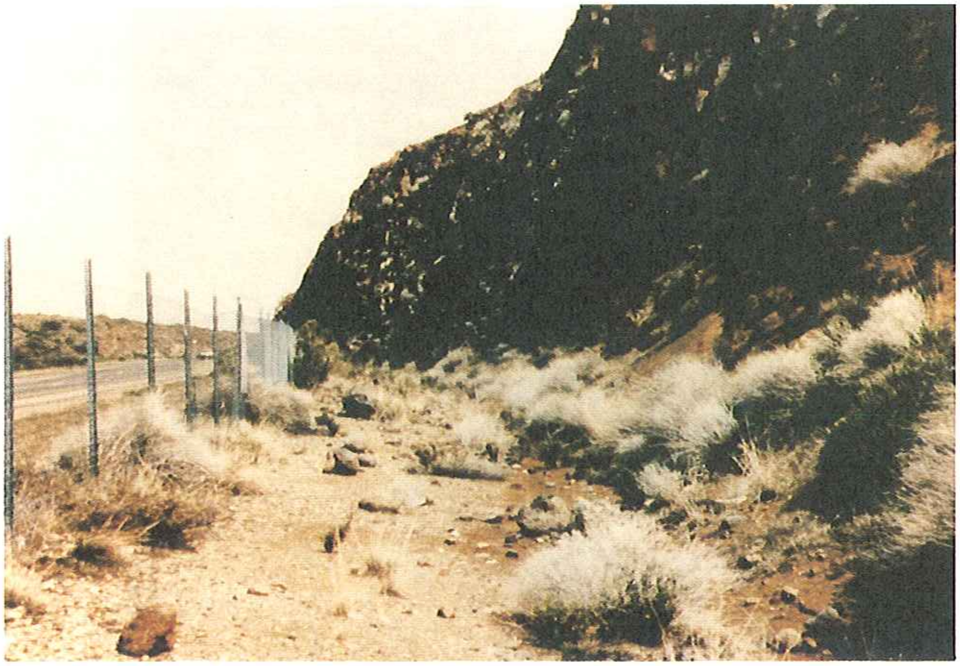


Figure 7-37. Ditch of more than adequate width but inadequate depth. The fine mesh fence would not be required if the ditch were deeper.

ROCK SL.	H(FT.)	W(FT.)
NEAR VERTICAL	20 - 30	12
	30 – 60	15
	> 60	20
0.25:1 or 0.30:1	20 – 30	12
	30 – 60	15
	60 – 100	20
	> 100	25
0.50:1	20 – 30	12
	30 – 60	15
	60 – 100	20
	> 100	25

W.S.D.O.T. DITCH DESIGN FOR ROCKFALL AREAS-DESIGN A

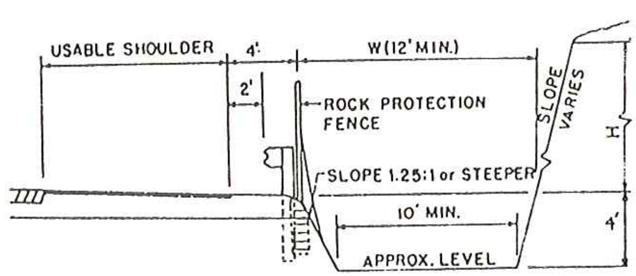
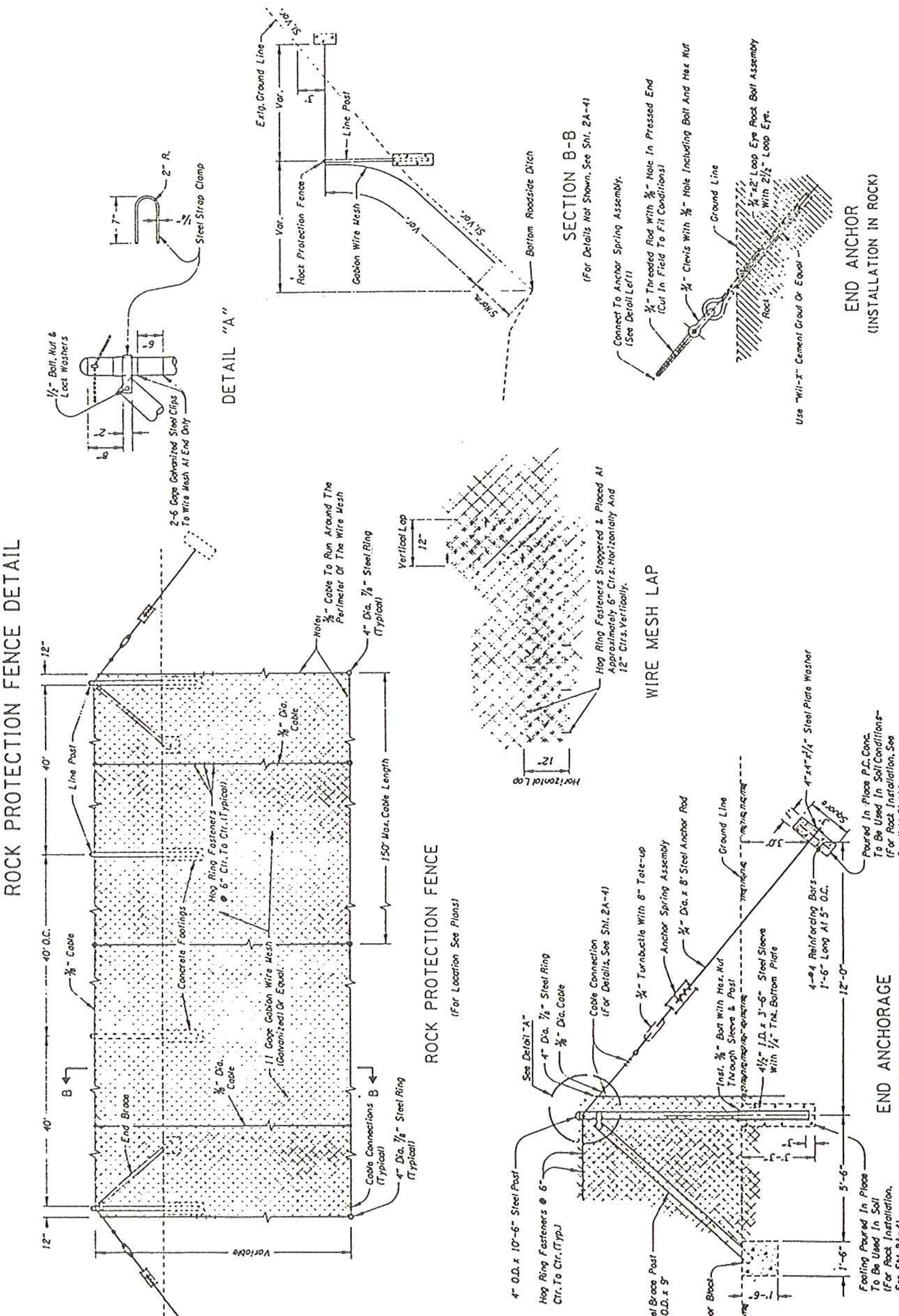


Figure 7-38. Typical Stage III ditch design. The State uses a staged approach depending on the severity of the rockfall problems *(Washington State Department of Transportation).*



ROCK PROTECTION FENCE DETAIL
ROCK PROTECTION FENCE
(For Location See Plans)
DETAIL "A"
SECTION B-B
(For Details Not Shown, See Sht. 2A-4)
WIRE MESH LAP
END ANCHOR
(INSTALLATION IN ROCK)
END ANCHORAGE
(For Details Not Shown, See Shts. 2A-2 And 2A-4)
Line Post
End Brace
Concrete Footings
Hog Ring Fasteners @ 6" Ctr. To Ctr. (Typical)
11 Gage Gabion Wire Mesh (Galvanized Or Equal)
Cable Connections (Typical)
4" Dia. 7/8" Steel Ring (Typical)
150' Max. Cable Length
Steel Strap Clamp
Rock Protection Fence
Gabion Wire Mesh
Bottom Roadside Ditch
Extg. Ground Line
Ground Line
Rock
Vertical Lap
Horizontal Lap
4" O.D. x 10'-6" Steel Post
Steel Brace Post 4" O.D. x 9'
Conc. Anchor Block
Anchor Spring Assembly

Figure 7-39

7.4.7. Rock Protection Fences and Nets

Many states have utilized flexible rock protection fences above the highway on the slope, near the toe of the slope or on the inner shoulder of the highway. Fences are considered feasible to catch small to moderate size rock with impact energies as high as 25 foot-tons. They are flexible and absorb the energy better than rigid barriers. Many States have experimented with various materials and design. Early catch nets were constructed of chain link or gabion-type mesh. More recently, cable type mesh, such as produced by Brugg (figures 7-40 and 7-41) and L'Enterprise Industrielle (figures 7-42 and 7-43) has been developed and tested successfully (Smith and Duffy, 1990).

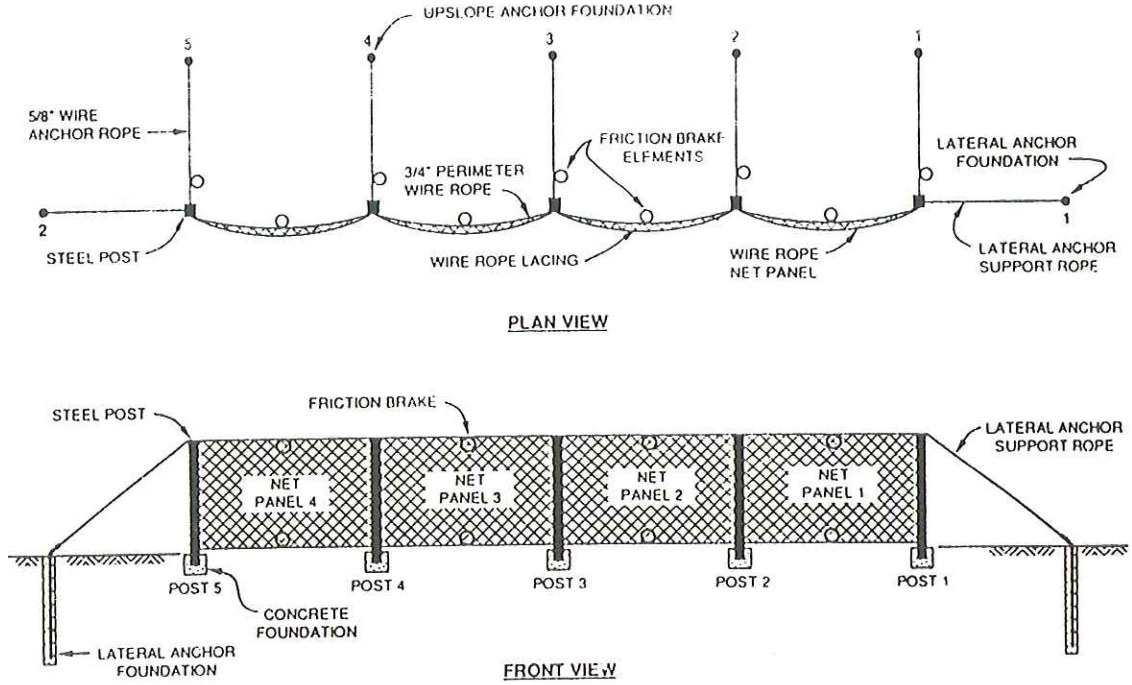
Note that the term fence is commonly used to describe a system that uses steel wire mesh like chain link or double twist hexagonal mesh. The term net is commonly used to describe a system that uses wire rope mesh.

Several major studies of rock net systems by Caltrans and others indicate that rock nets are viable to absorb and dissipate rockfall impact energies as high as 400 ft-tons. Rock rolling tests have been performed on a variety of slopes from 34-45 degrees and 60 to 500 feet long. Rock weights ranged between 300 and 13,000 lbs. More than 200 rock have been rolled into the nets during these tests.

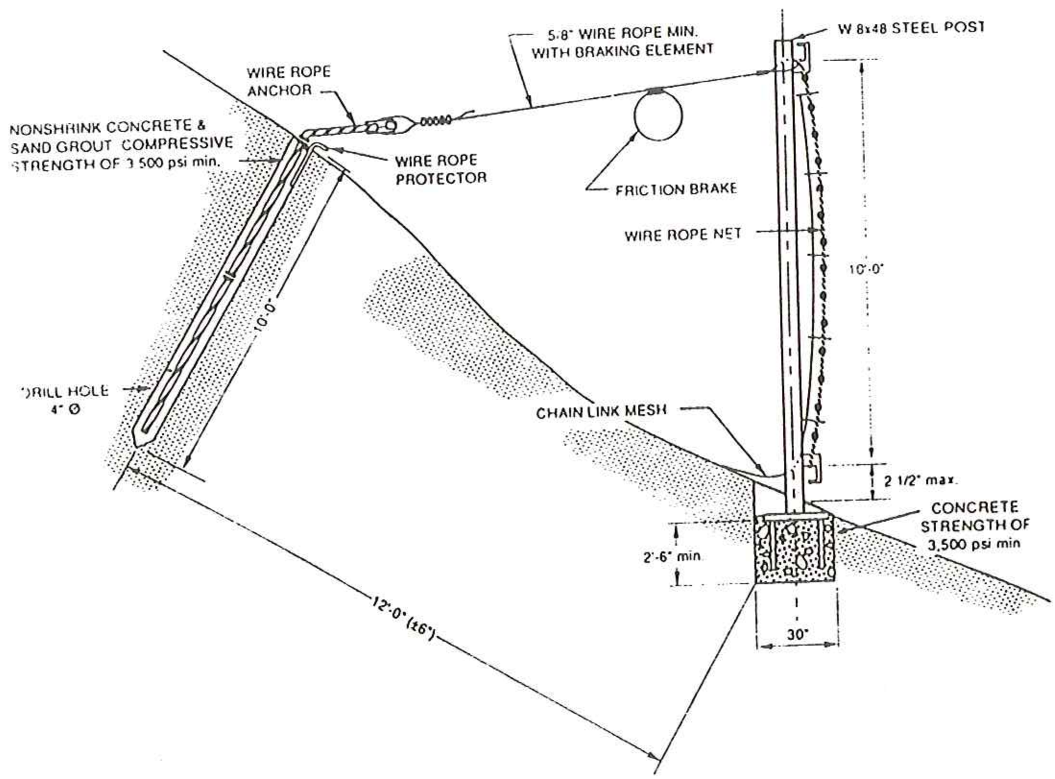
Energy and rockfall trajectory are the most important design considerations for successful rock fence and net design.

Maintenance and cleaning of the nets were easily accomplished with Caltrans maintenance personnel. Removal of rockfall debris is accomplished by raising or lowering the net to allow access. Nets at road level can be cleaned with normal maintenance equipment. Damaged net components can usually be reused or repaired in a short time by maintenance staff.

Wire rope restraining fences and nets are now an integral part of the rockfall mitigation measures in many states. Testing is continuing to develop stronger and more efficient systems.



PLAN VIEW AND FRONT VIEW OF BRUGG ROCK NET CONSTRUCTED FOR THE INITIAL FIELD TESTING



SIDE VIEW OF A BRUGG BASEPLATE FOUNDATION WITH UPSLOPE ANCHOR SUPPORT
All dimensions are site specific.

Figure 7-40. Brugg Rock Net (Smith and Duffy, 1990).

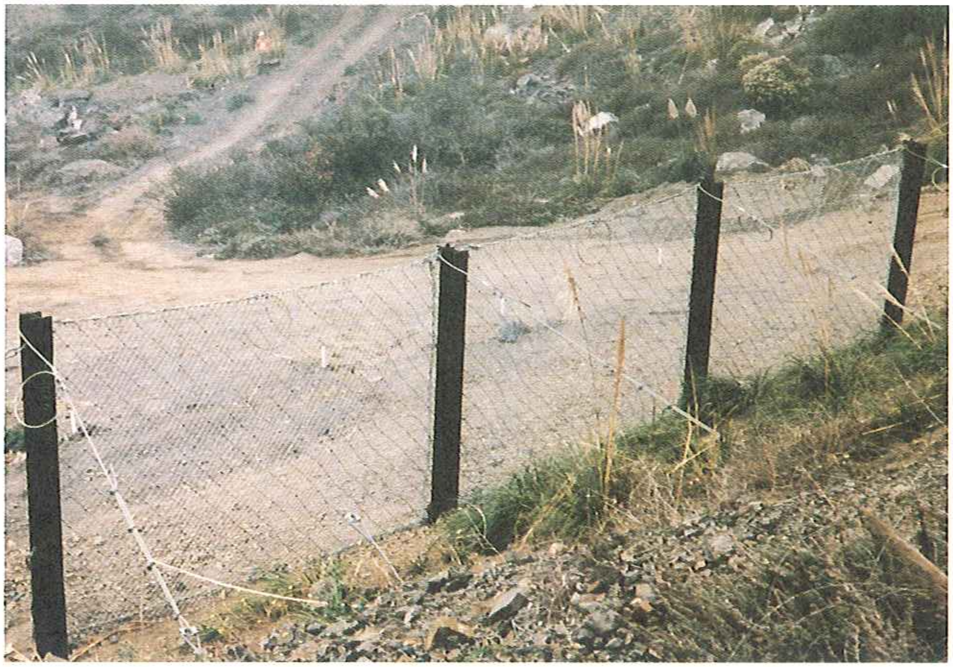
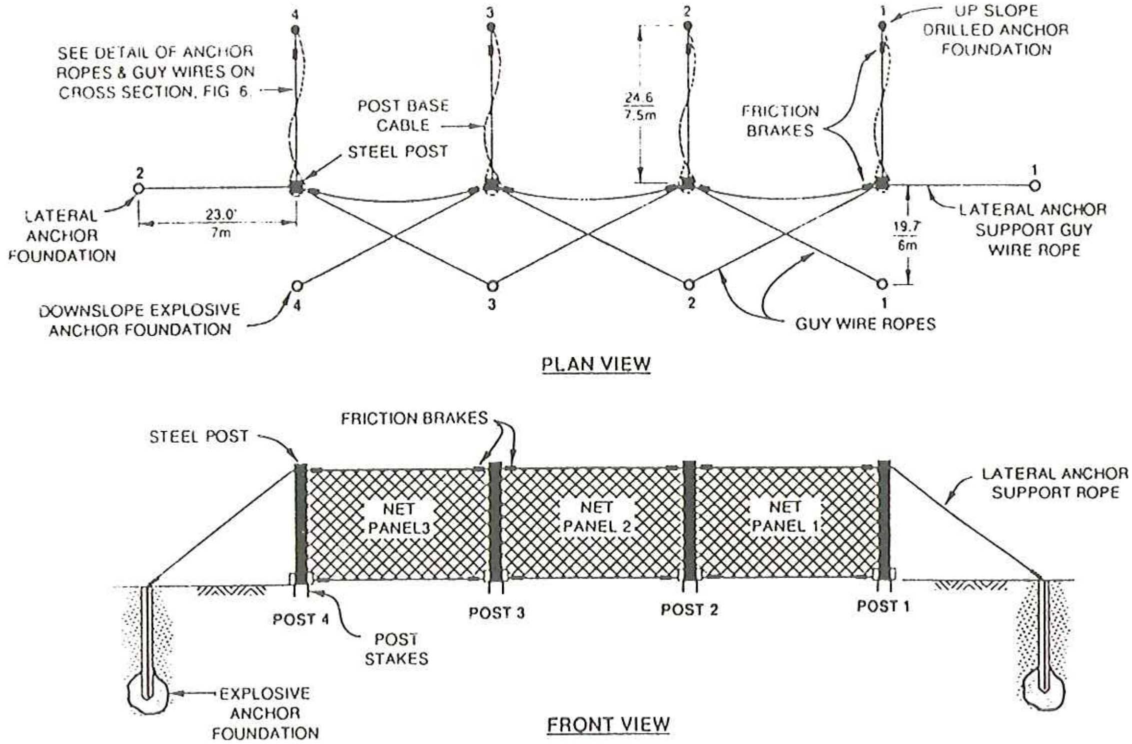
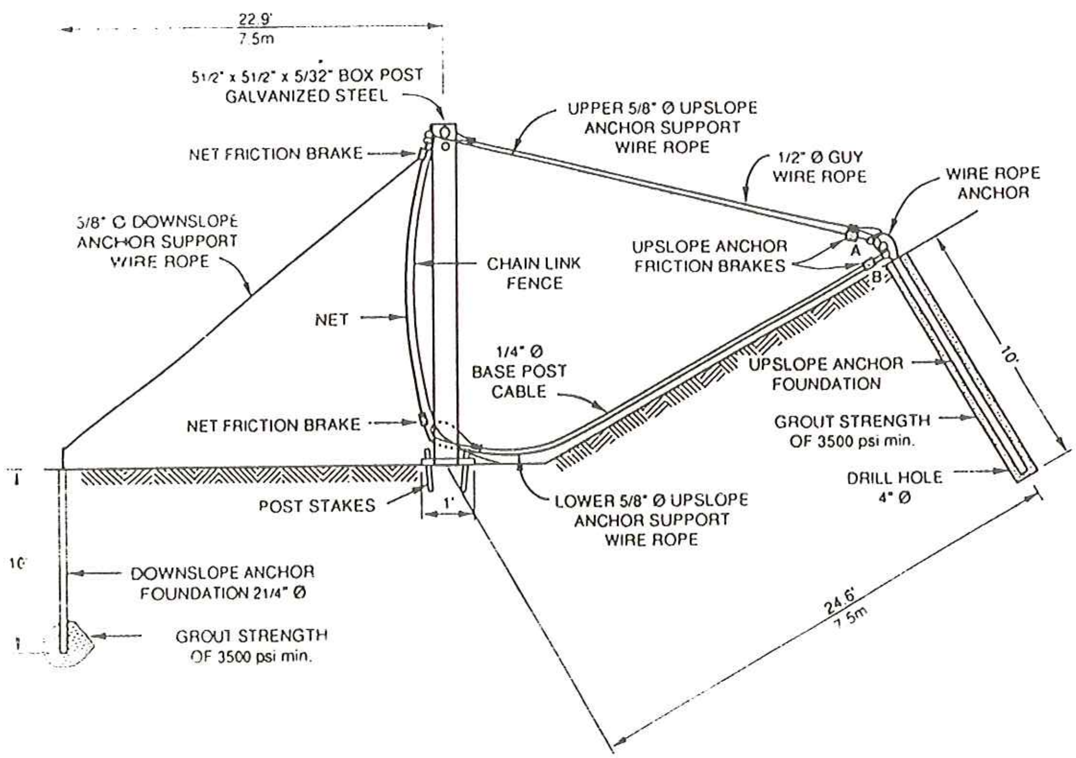


Figure 7-41. Brugg Rock Net installed with cables to a buried anchor *(Courtesy Caltrans)*.

Figure 7-42. L'Enterprise Industrielle Rock Net installed. This fence is about 12 feet (3.66 meters) high. Both upslope and downslope anchors are used *(Courtesy Caltrans)*.



FRONT AND PLAN VIEW OF THE INDUSTRIAL ENTERPRISE ROCK NET CONSTRUCTED FOR FIELD TESTING



SIDE VIEW OF THE INDUSTRIAL ENTERPRISE ROCK NET
All dimensions are site specific.

Figure 7-43. L'Enterprise Industrielle Rock Net (Smith and Duffy, 1990)

Colorado has developed a flexible rock catchment fence called the "Flexpost Fence". The post bends at the hing point near the base. This allows the fence to redirect the rock to the ground where the energy is dissipated. For rock sizes of three feet and smaller, the flex spring action will cause the fence post to return to a vertical position (figures 7-44, 7-45 and 7-46). The fence was field tested by the Colorado Department of Transportation and a design load of 20 foot-tons was established. In cooperation with the University of Colorado, a computer program was developed to analyze the interaction of fence and rock impact. With this model, a 40 foot-ton capacity fence has been designed.

7.4.8. Cable Anchored Hanging Fences

At numerous locations, rock ravelling is confined to a narrow channel or gully. Where the channel is devoid of soil or vegetation, the velocities can become high and the rock can bounce to moderate height. For high rock energies, woven wire rope and submarine netting have been successful.

Draped wire mesh is strung across the rockfall track hanging from a strong cable tied to trees or grouted eye hooks. The cable is strung 10 to 20 feet (3.5 to 6.1 meters) above the channel so high bouncing rocks will be caught. Rock hits the mesh and its velocity is reduced or the rock is stopped. Maintenance crews must remove rock that is caught. Where large numerous rock may fall, mesh may be hung from several cables strung across the channel at various elevations. Logs may be tied to the bottom mesh in an attempt to stop the rock at the lowest draped mesh Figure 7-47 shows such a multiple draped mesh installation.

7.4.9. Rockfall Barriers and Walls

Walls of many designs have been used to stop rockfall from encroaching on the highway. Typical installations include Jersey barriers (figure 7-48), gabion basket walls (figure 7-49), concrete lock block walls (figure 7-50), concrete retaining walls, combination walls (figure 7-51) and gravity walls below wire mesh above (figure 7-52). In some instances, steel H piles with timber lagging and precast concrete lagging have been used (figure 7-53).

Figure 7-44. Flexible post type of fence developed by the State of Colorado to catch ravelling rock. The posts bend on impact. Where the rock caught is small the fence springs back into place *(Courtesy Colorado Department of Transportation).*

Figure 7-45. Prestressed wire strands that act as a spring inside the lower portion of the post *(Courtesy Colorado Department of Transportation).*

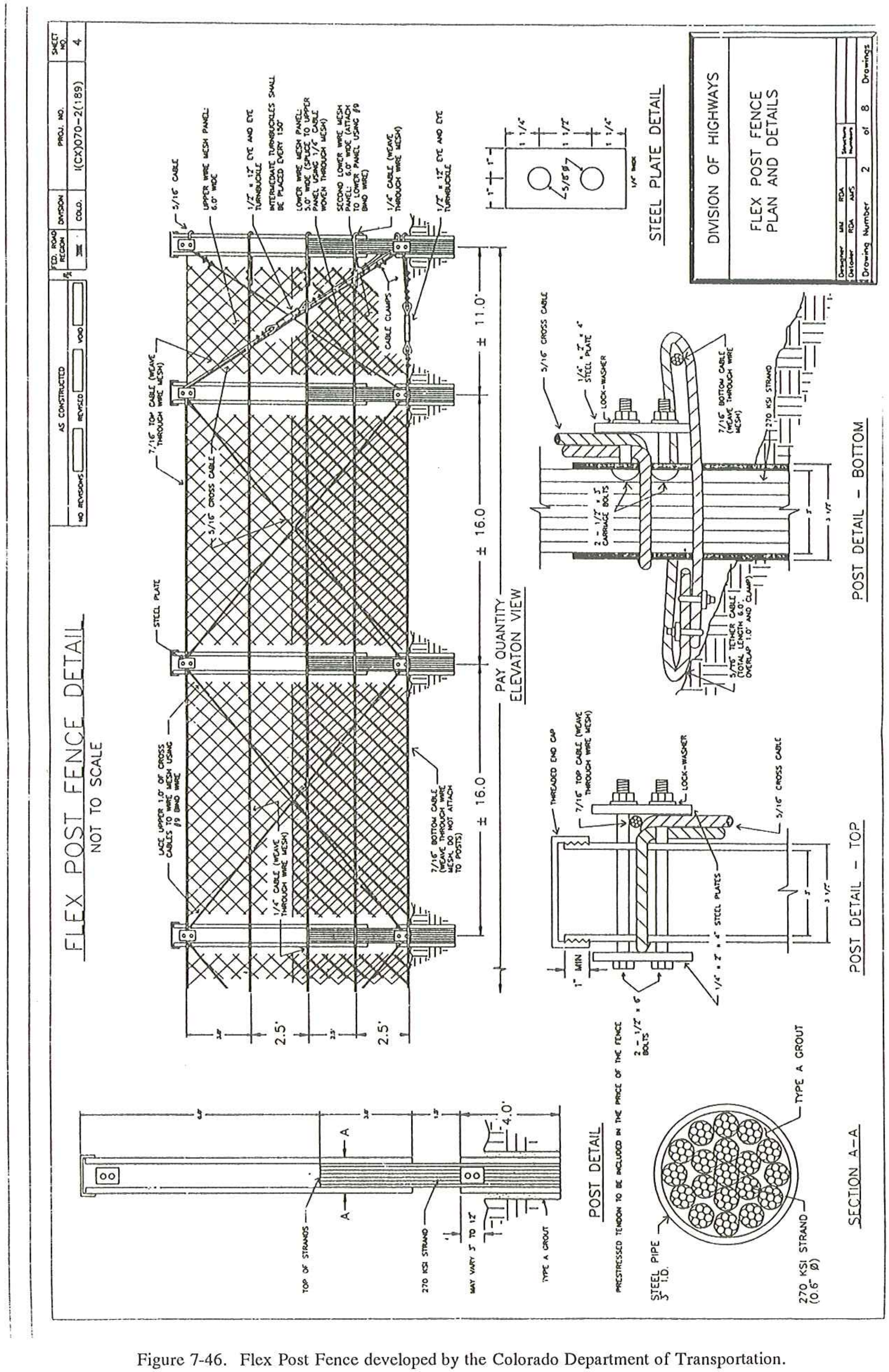


Figure 7-46. Flex Post Fence developed by the Colorado Department of Transportation.

Figure 7-47. Multiple draped wire mesh draped from anchor cables. This multiple system is designed to intercept a large volume of rock.

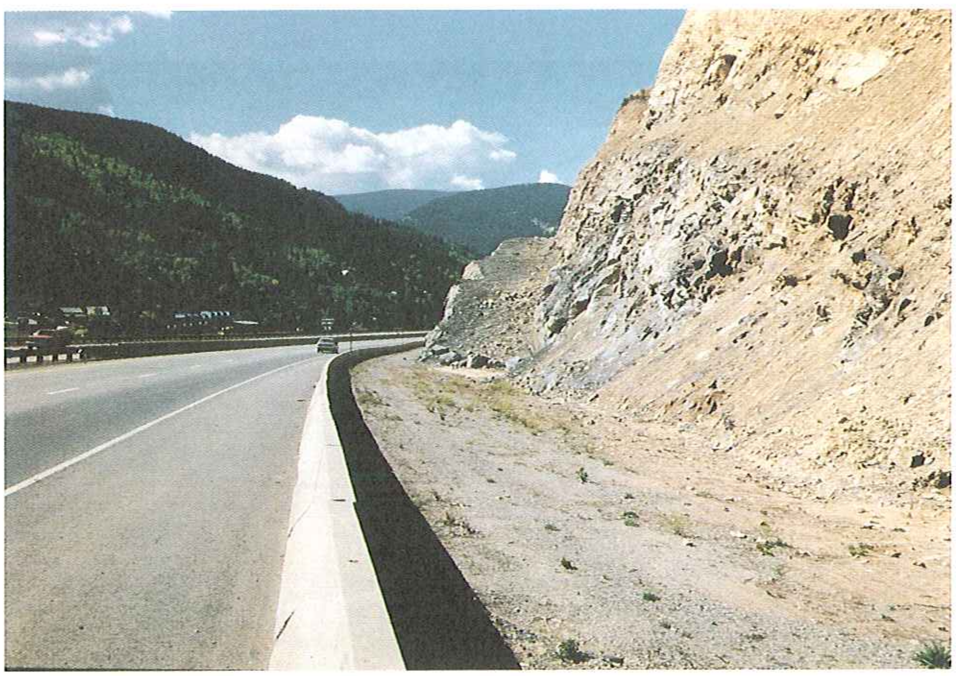


Figure 7-48. Jersey barrier along the outside shoulder to catch rolling rock.

Figure 7-49. Gabion wall along inner shoulder used to catch ravelling rock. When cleaning the ditch care must be taken that the equipment does not break the gabion wire.

Figure 7-50. Concrete lock block wall above the highway installed to catch rolling rock behind. Note the attractive face texture.

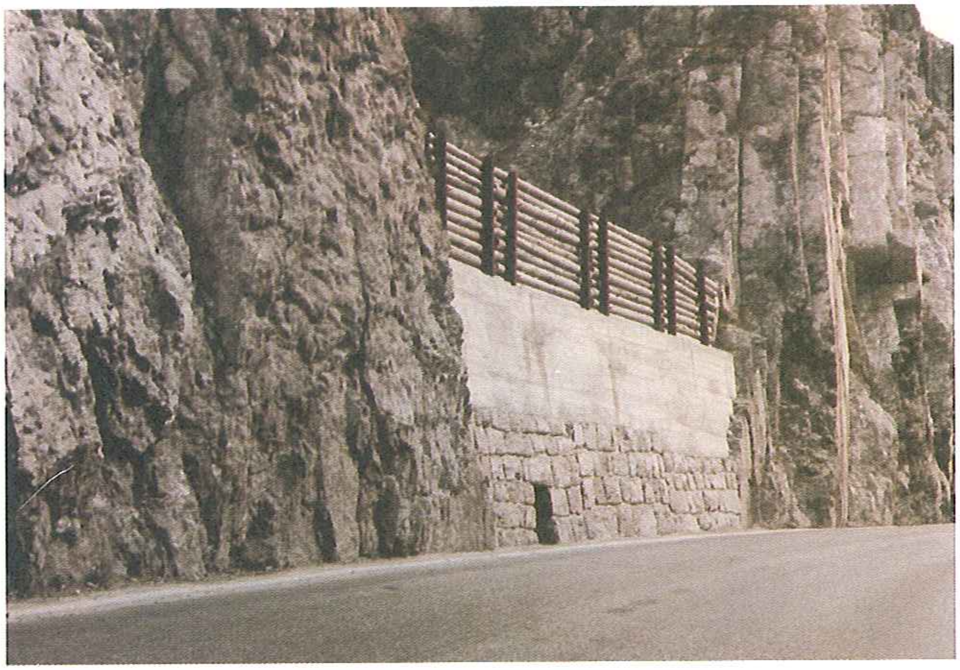


Figure 7-51. Combination catch wall on a road in Switzerland. The wall height has been increased as the volume proved inadequate because of not cleaning behind the wall.

Figure 7-52. Wire mesh fence above gravity base wall. Cabled mesh covers the rock face. Photo south of Tokyo, Japan.

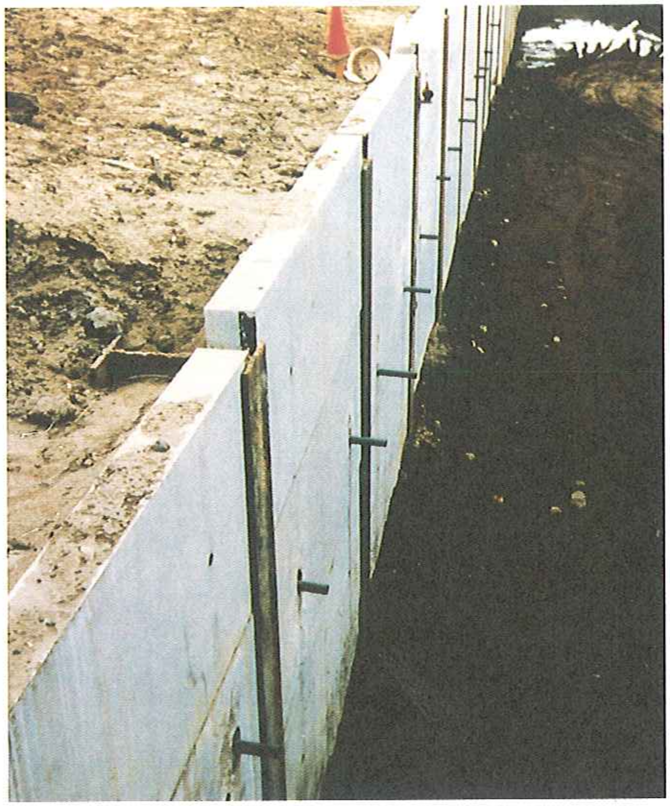


Figure 7-53. Steel piles with precast lagging-type wall.

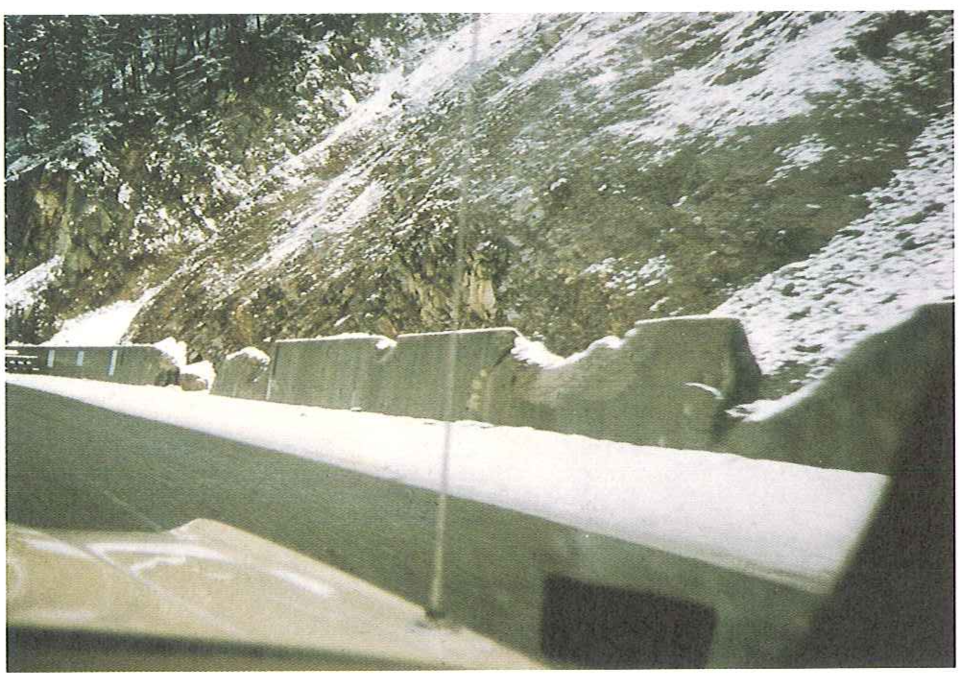


Figure 7-54. Rigid concrete wall badly damaged by large ravelling rock. Flexible-type walls resist impact better than rigid walls.

The walls are usually positioned on the inner shoulder so that they increase the catchment capacity of the ditch.

The most common shoulder catch wall in the U.S. is the Jersey barrier. However, experience has shown that rigid walls have a tendency to break under high-impact loads (figure 7-54) and these small-sized barriers will not stop large-size high-energy rockfalls. In some instances, they have fences attached above.

Walls with vertical back faces are best since any rock that hits this face cannot overtop the wall. It is essential that the ditches periodically be cleaned of rockfall or falling rock can bounce off rock in the ditch and onto the highway.

Colorado has recently experimented with a geosynthetic reinforced wall which uses the timber facing-forming methodology developed by the Colorado Transportation Institute (figure 7-55, and 7-56). The double-sided test wall was 10 feet high and 6 feet thick. Progressively larger rocks were rolled into it until "significant" damage was incurred with an impact of 500 foot-tons. The wall remained functional but required repair. Thicker walls are capable of stopping larger rock energies.

7.4.10. Rock Sheds and Tunnels

The use of tunnels or rock sheds for rockfall protection are warranted only when other methods of stabilization and protection are deemed ineffective and the cost can be justified. They are expensive, but will provide complete protection.

A rock shed must withstand the energy of the largest rock mass likely to pass over it during its life. A cover of loose sand on the structure can reduce the stresses induced during impact. The top angle can be varied to reduce the impact. Figure 7-57 shows three rock sheds over a railway. Two of the rock sheds are timber and one is concrete. Timbers used should be pressure treated to reduce decay and be fire resistant. Design and location of the upper chute is a critical feature of the design. Figure 7-58 shows a precast concrete roof shed.

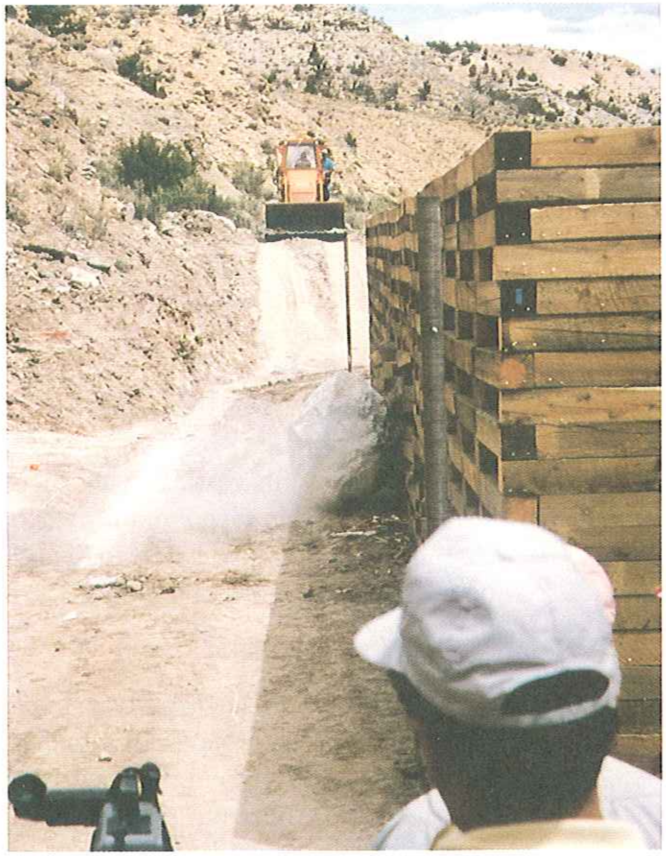


Figure 7-55. Geosynthetic reinforced impact timber crib wall being tested for strength and durability *(Courtesy Colorado Department of Transportation).*

Figure 7-56. Result of a 800 foot-ton rock impact. The geosynthetic rockfall barrier was damaged but remained functional *(Courtesy Colorado Department of Transportation).*

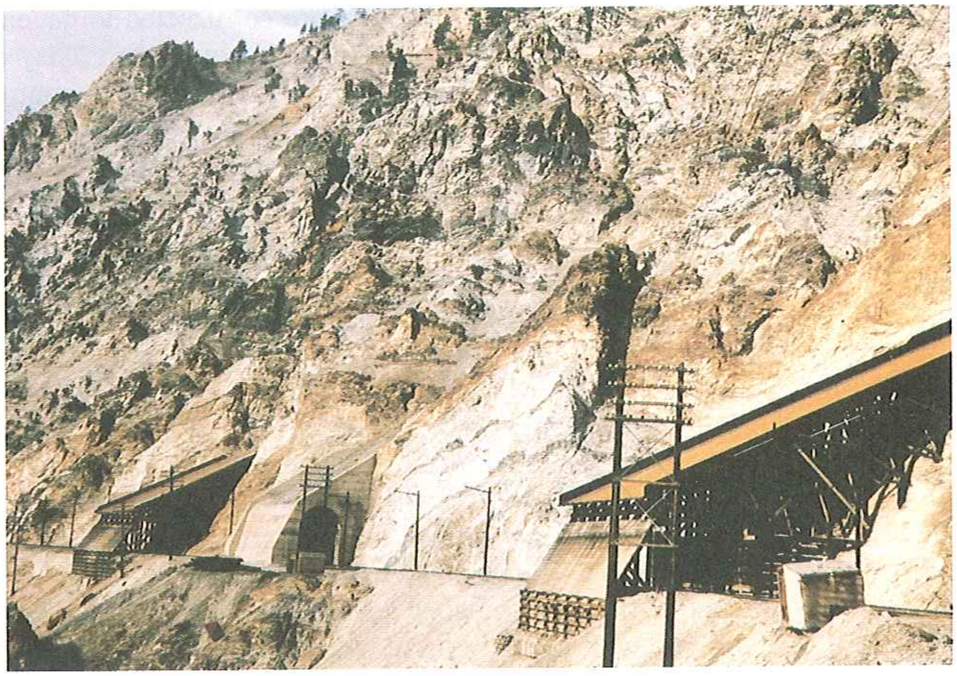


Figure 7-57. Concrete and timber rock sheds to carry ravelling rock over the Canadian National Railway near Lytton, B.C.

Figure 7-58. Precast concrete rock shed constructed to protect the highway.

Tunnels usually are constructed to develop good alignment in mountainous canyon areas. They present positive rockfall protection. A most important design feature is to extend the portal far enough beyond the rock face so rockfall from above the portal will be contained on the roof top. Figures. 7-59 and 7-60 show two tunnels in rock.

7.5. WARNING OF POTENTIAL ROCKFALL

The courts in some States have taken warning signs into consideration when reviewing potential liability, particularly when a vehicle strikes rock on the highway. The interpretation is that the driver should use extra caution in driving through the area covered by the warning. This would include driving slower than the normal speed limit. If the driver does not exercise extra caution, State liability will normally be reduced.

The wording of the warning will also be important. A general sign, such as shown in figure 7-61 will carry less impact than a specific sign, such as shown in figure 7-62.

Where rock falls on a vehicle, a sign has negligible impact on liability.

Warning systems have been used for decades by North American railroads. Such a system is shown in figure 7-63. Multiple wires are strung across the rockfall pathway. These are electrically connected to a warning system, usually tied into the railway dispatcher's office. If a rock should break a wire, the dispatcher is alerted and any trains in the area are immediately warned by radio. Maintenance forces are also alerted and dispatched to the warning location.

Such a system could be used above highways, particularly below cliffs that are well above the highway. The most effective control would be to activate and lower a traffic control arm on either side of the rockfall path, such as is common in snow avalanche areas.

It is obvious, however, that a stabilization or catchment program above the highway is preferred to relying on a warning system approach.

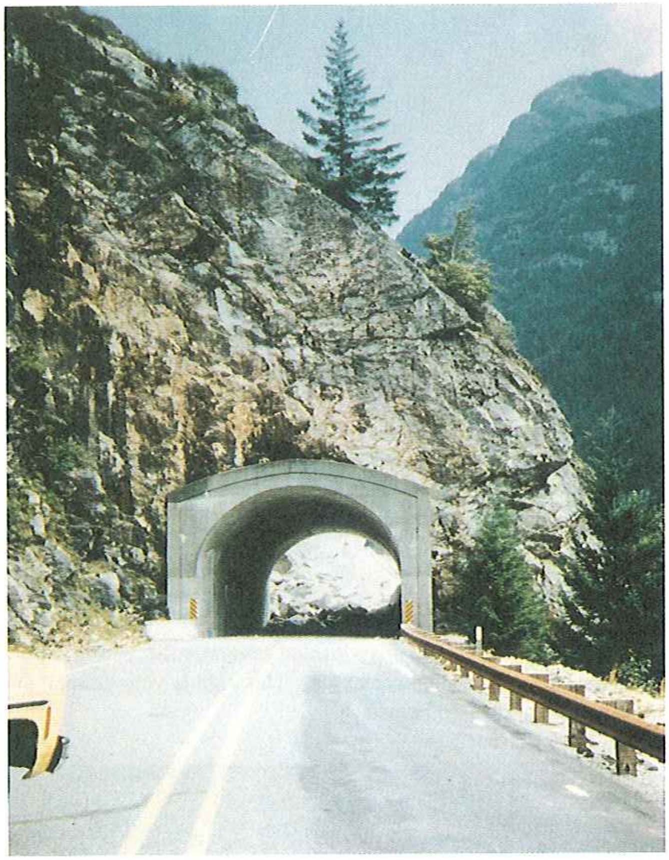


Figure 7-59. Concrete-lined tunnel in rock. The portal must extend out from the rock face a sufficient distance to ensure rock does not fall over the portal onto the highway *(Courtesy Washington Department of Transportation).*

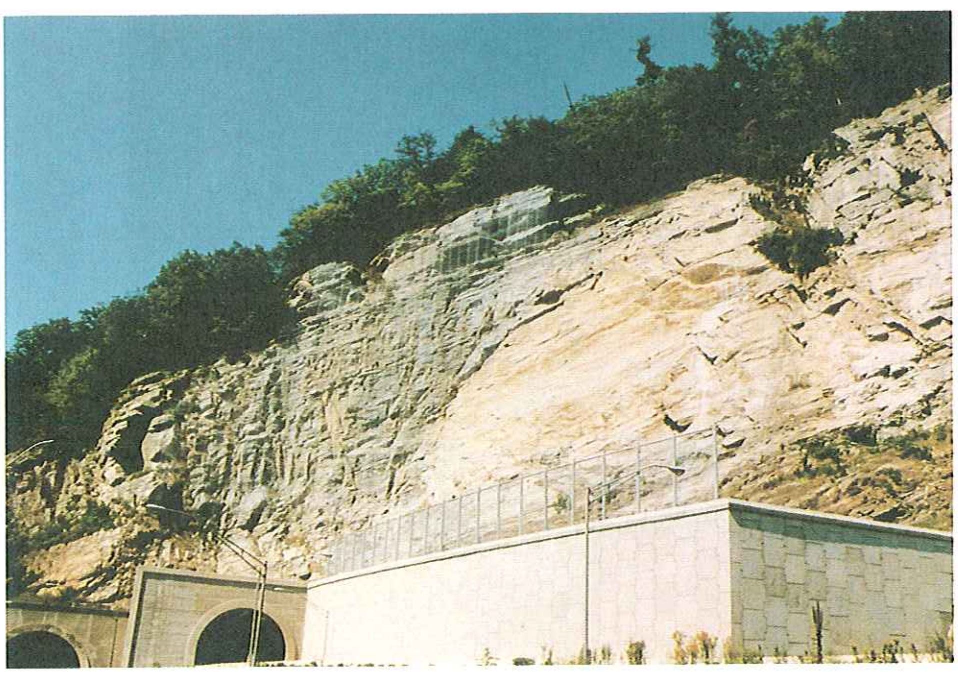


Figure 7-60. Double concrete tunnels. The portals extend well out from the face. A rock catchment area has been extended below the rock slope beyond the tunnel by use of a retaining wall *(Courtesy North Carolina Department of Transportation).*

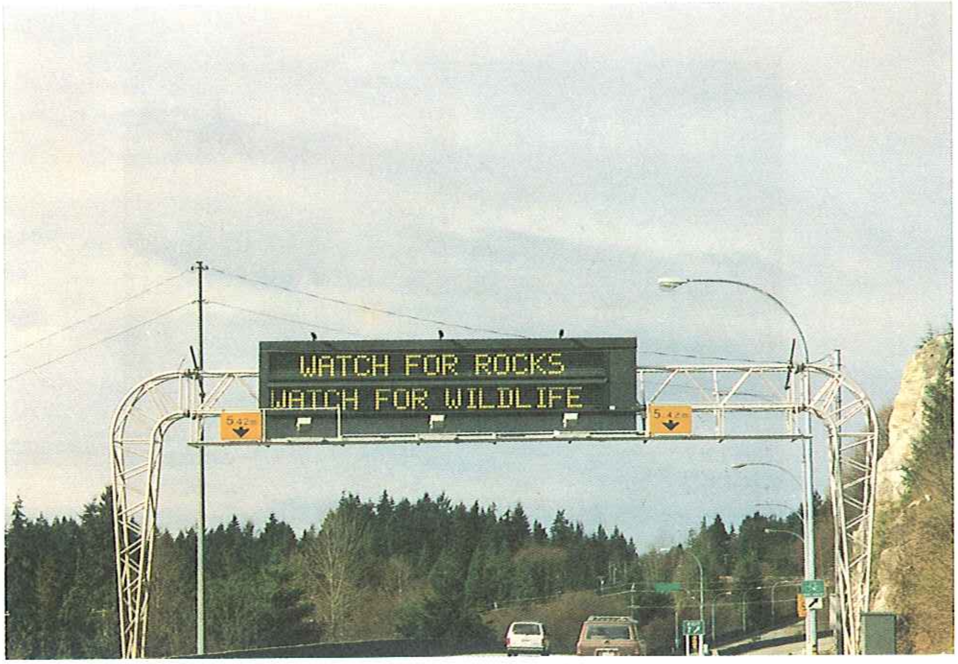


Figure 7-61. Overhead warning display located several miles before a 30-mile (48.3Km) stretch of highway with many high rock cuts. This sign is very general and has less impact on a driver than the sign below.

Figure 7-62. Warning sign showing rock hazard ahead. This sign is specific and should alert the driver to potential rockfall.

Figure 7-63. Electrified wire warning fence located below a rockfall path adjacent to the Canadian Pacific Railway near Revelstoke, B.C. Such a warning fence could also be located to warn of rockfall above highways.

When considering a warning fence, three recommendations should be considered:

- Choose the wire spacing according to rockfall sizes experienced at the site.

- Provide a catchment ditch behind the warning fence.

- Eliminate the lower wires so the rockfall can be removed.

It has been documented that the wire fences possesse a low efficiency with more than 50 percent of the alarms found to be false.

Another effective warning method consists of anchoring a single wire on an unstable rock mass or rock or across a rock slope above the right-of-way. The wire is then attached to a signal.

Laser shields are an electronic method of protection; they are set up and act similar to the electric wire fence, but they eliminate the maintenance problem associated with wires. However, the system is expensive and is not noted for durability in extreme climates.

Monitoring also can provide an effective means of rockfall warning. Typical procedures are described in chapter 5.

7.6. ROCKFALL CONTROL DURING CONSTRUCTION

Many projects involve highway realignment and reconstruction in areas of rock. Much of this work must be completed under active traffic conditions since the highway is frequently in a confined topographic location.

Wherever traffic detours are practical they should be implemented. Construction costs under traffic conditions are many times more expensive than when no traffic is involved.

Where traffic volumes are light to moderate, traffic can be controlled through the construction area so that work can proceed between traffic stoppages. Traffic control is performed using flagpersons equipped with two-way radios; they are in constant contact with the construction project superintendent.

A wide variety of means to control rockfall under traffic have been used. The primary objective is to stop any rock from reaching the travelled roadway.

Figure 7-64 shows a moveable timber-guard rail-fence combination being pulled by a truck crane from one location to another site. Heavy-duty portable timber frames have also been used. These are suitable where the rockfall energy is low to moderate and where there is room on the shoulder for the rock catchment unit.

The State of North Carolina has utilized a unique rockfall catchment system. It has placed wrecked automobiles along the outer traffic lane to catch construction rockfall (figure 7-65). One traffic lane was taken over to place the vehicles.

If large volumes of rock require blasting for removal, it is essential that the blasting be well controlled with small-diameter closely spaced holes, small charges per hole, and multiple delays. The blast should be initiated from one end of the cut and not parallel to highway. Traffic should be stopped from 5 minutes prior to the blast until 10 minutes after the blast.

The State of Oregon has used baled hay to act as a catchment (figure 7-66). The hay is inexpensive, quick to place, and provides good cushioning resistance. Lateral resistance must be provided for the hay. Jersey barriers will provide resistance where a guard rail does not exist.

Jersey barriers have commonly been used to protect against rockfall. They are effective where small rock exists and where the bounce height is low. Where large volumes occur, they are not suitable. Figure 7-67 shows the inadequacy of a standard size Jersey barrier against a large rockfall.

Larger size Jersey barriers can be used to catch larger volumes or to prevent rock from rolling or bouncing completely over the roadway and onto private property or environmentally sensitive areas (figure 7-68). In this case, the barrier is placed on the outer shoulder. These can be used as portable barriers during construction or as long-term permanent barriers.

The New York Thruway Authority has developed a portable rockfall catchment fence. One lane of traffic is taken out of service. At some locations an additional traffic lane can be developed by using the shoulder of the outer two-lane section.

Figures 7-69 to 7-72 illustrate the design and application on a rock stabilization section north of New York City.

The design involves precast concrete base sections with steel post and cable mesh sections. The photos show a controlled blast that was detonated with considerable rock reaching the roadway. The freeway was closed during the blast and opened as soon as the roadway was cleared in a matter of minutes.

Figure 7-64. Moveable rock catchment guardrail and fence to catch smaller size rock. The unit is being moved to a new construction site. Moderate shoulder width is required *(Courtesy Colorado Department of Transportation).*

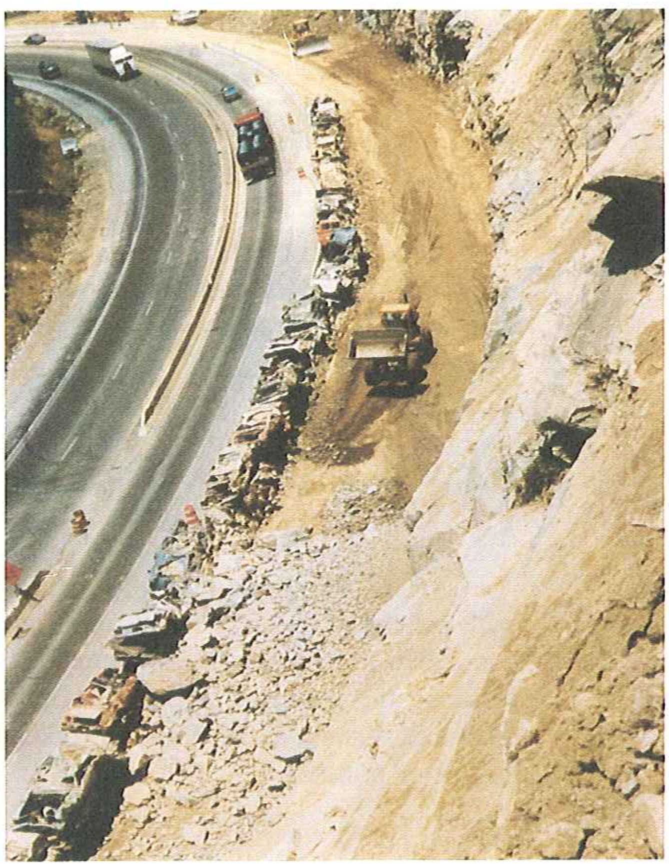


Figure 7-65. Wrecked car bodies placed on the outer shoulder to catch rockfall from above. The loader is removing the blasted rock. One lane of freeway has been closed *(Courtesy North Carolina Department of Transportation).*

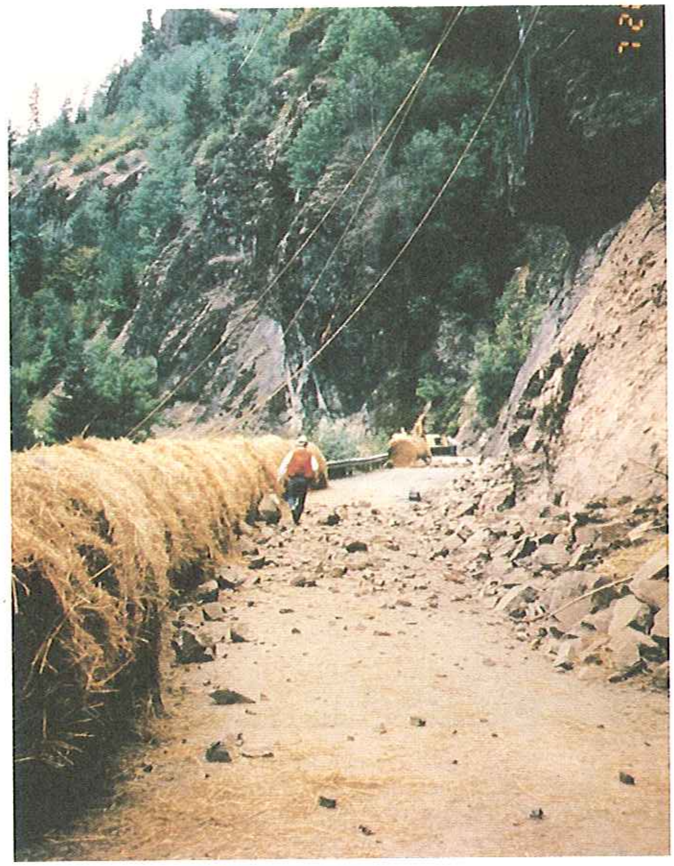


Figure 7-66. Round bales of hay placed against the inner guard rail provide protection during construction against falling rock. The hay provides good impact cushioning *(Courtesy Oregon Department of Transportation).*

Figure 7-67. The standard size Jersey Barrier was inadequate to contain the large rockfall *(Courtesy North Carolina Department of Transportation).*

Figure 7-68. Oversized Jersey barrier placed on the outer side of the roadway to keep construction rockfall from entering an environmentally sensitive rafting river below.

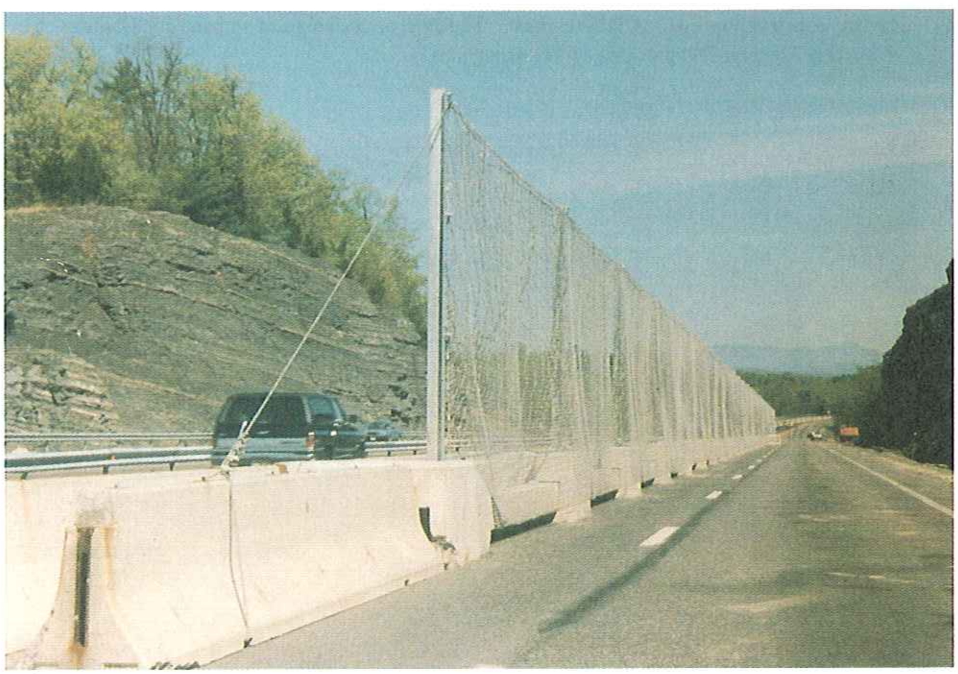


Figure 7-69. Precast portable rockfall fence installed adjacent to a rock stabilization project before construction. All components can be moved and used elsewhere *(Courtesy New York State Thruway).*

Figure 7-70. Controlled blast to remove unstable rock with the portable rock catch fence in place *(Courtesy New York State Thruway).*

Figure 7-71. Road cleanup after the blast. Note the few small rocks that overtopped the fence *(Courtesy New York State Thruway).*

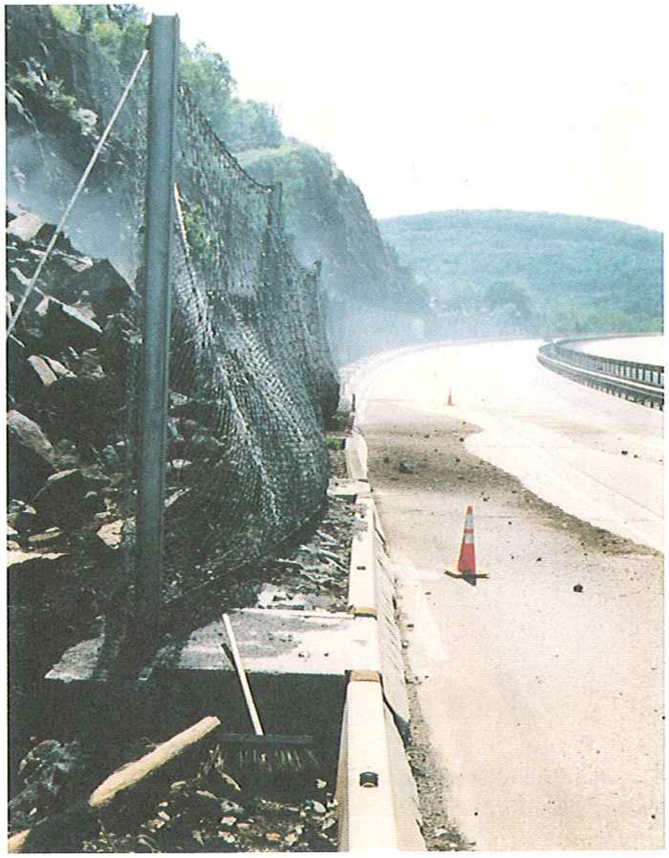


Figure 7-72. Closeup of the cable mesh appearance and behavior after the blast. Note the small rock and gravel sizes on the roadway. The combination of catchment width, fence, and barrier was effective *(Courtesy New York State Thruway)*.

CHAPTER 8

AESTHETICS AND SAFETY CONSIDERATIONS

8.1. INTRODUCTION

Most highway projects for new construction or reconstruction must pass scrutiny and obtain permits from other agencies prior to approval and construction.

Some of these agencies have attempted to pressure highway departments into incorporating aesthetic features that they feel will provide "a more natural appearance" without adequate recognition of costs and safety. Typical proposals are:

- Avoid uniform smooth rock cut faces, for example excavate the slope to emulate natural rock out croppings (a sculptured look).
- Avoid visible blast hole scars on the slope face.
- Create isolated planting benches on the slope face.
- Design rock slopes under the direction of a landscape architect.

The aesthetic concerns have not originated from the travelling public, who are the prime users. Discussions with senior engineers in numerous States that have many rock cuts have not indicated any public complaints regarding rock slope appearance. The primary concerns of the travelling public are safety, unimpeded travel, and minimum impact on tax dollars.

The public is conditioned to specific aesthetic quality and environmental control for parks, fishery areas, industrial complexes, and housing developments. They have also become conditioned to the highway environment. For example, the color of traffic control lines (white and yellow), the color, shape and location of traffic control and warning signs, the shape of Jersey traffic barriers, the design and material in metal guard rails, the use and design of rock catch fences, retaining walls, and culverts are accepted by the public as essential for safety reasons.

Many highways in the United States pass through mountainous terrain and require rock excavation faces that may extend upwards of 100 to 300 feet (30.5 to 91.5 meters). Long-term stability of these slopes must be maintained. In many areas, highways are located below and adjacent to natural high rock cliffs with or without talus slopes below. These slopes are continually ravelling because of natural causes. Transportation departments must strive to provide a much safer highway stability environment than nature provides.

One of the major rockfall safety concerns has been the damage created within the rock slope due to excessive blasting energy used to break the rock. Practically all rock slopes blasted prior to 10 to 15 years ago have experienced blast damage that resulted from uncontrolled blasting. In some instances, discontinuities are opened or new cracks have been formed up to 50 to 60 feet (15.3 to 18.3 meters) back into the slope. Where such uncontrolled blasting has been used, a long-term supply of rockfall is "built" into the slope with resultant high maintenance costs and safety hazards to the travelling public (Chassie 1992).

Rockfalls and small rock slides have caused traffic delays, accidents, injuries, and deaths in many States. Figures 8-1 to 8-4 are examples of accidents caused by rockfalls. Cleanup, maintenance, and litigation costs run into the hundreds of millions of dollars. As a result, transportation departments have generally established that the top design and construction priority is ***SAFETY***.

In fact, as of this writing, the law governing the United States Federal-Aid highway program (Title 23 U.S. Code and the 1991 Intermodal Surface Transportation and Efficiency Act) mandate the following:

- "It shall be the National Policy to bring all of the Federal-Aid systems up to standards and to increase the safety of these systems to the maximum extent."

- "The secretary (of Transportation) shall not approve plans and specifications for proposed highway projects if they fail to provide for a facility that will adequately meet the existing and probable future traffic needs in a manner conducive to safety, durability and economy of maintenance."

Figure 8-1. Rockfall near Icicle Cliffs, Idaho. A car hit the rockfall at night and went off the road into the adjacent river. Three passengers were killed *(Courtesy Idaho Department of Transportation).*

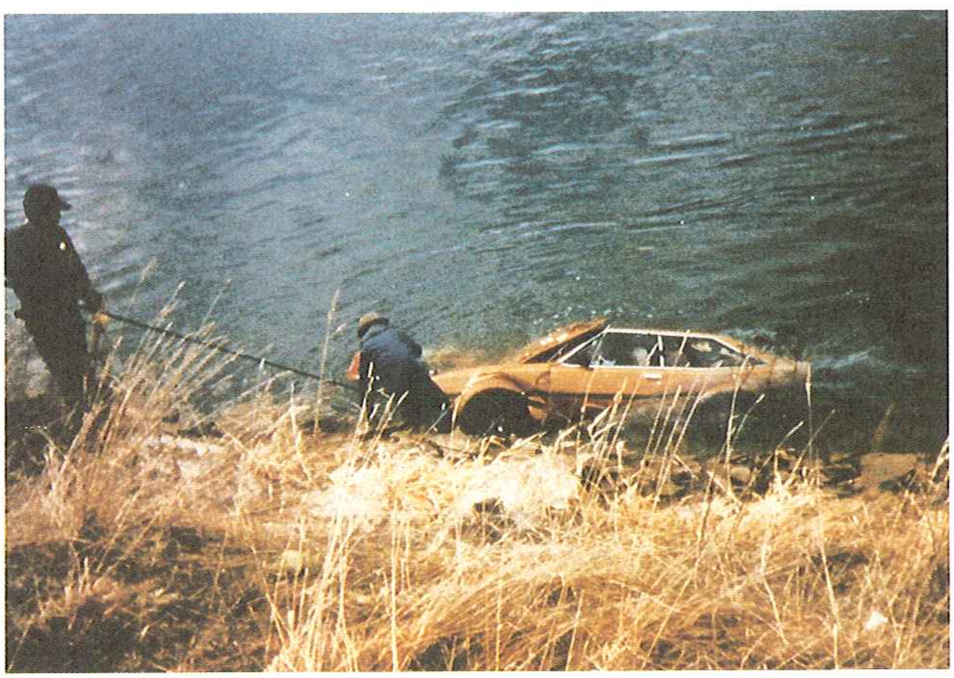


Figure 8-2. Auto being removed from the water *(Courtesy Idaho Department of Transportation).*

Figure 8-3. Large rockfalls blocking the highway. The nose had to be blasted to clear the train. Columbia River Gorge, Washington *(Courtesy Washington Department of Transportation)*.

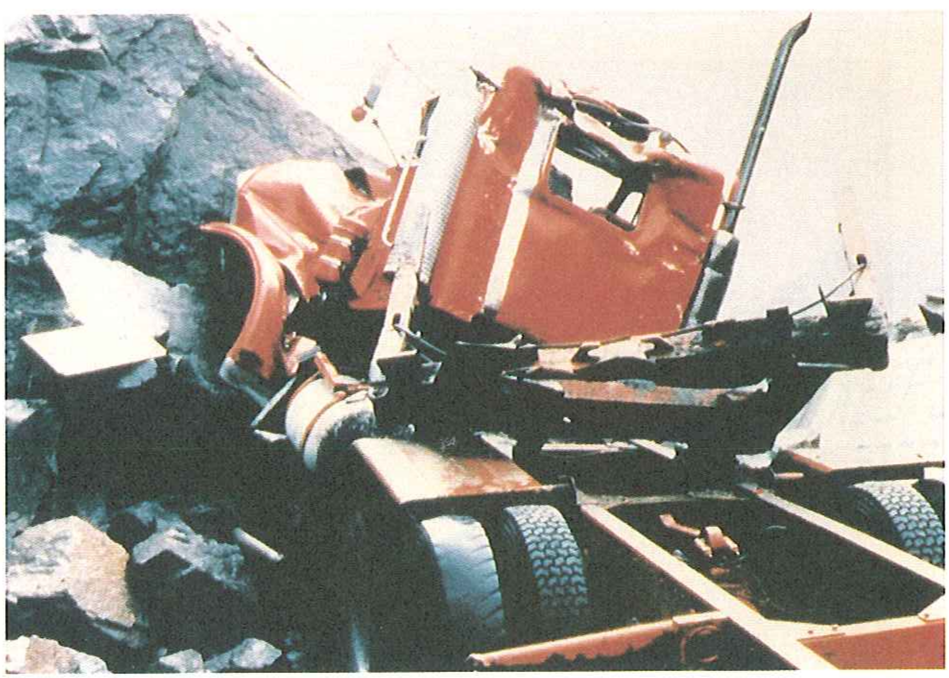


Figure 8-4. Truck that rammed a large rockfall. The driver was seriously injured. Columbia River Gorge, Washington. A lawsuit was settled for $175,000 *(Courtesy Washington Department of Transportation)*.

- The legislation states that the intent of the previous requirement is to "minimize safety hazards and insure that public funds will not be squandered in a demonstrably unsafe proposal."

In order to minimize the rockfall hazard many design and construction techniques have been developed (chapter 7). Procedures used to reduce rockfall hazards along railway lines have added to this experience (Brawner and Wyllie 1975, Brawner 1978 and Peckover 1975).

8.2. EARLY ROCK SLOPE DESIGN ERRORS

Some procedures that were used for many years are considered today to be poor practice. For example, for many highway agencies, the standard specified slope angle in rock was ¼H:1V. This gave no consideration to the rock characteristics or orientation or dip of the geologic structural discontinuities in the slope. Where they dipped out at angles flatter than ¼:1 and steeper than the effective angle of friction of the discontinuities, rockfalls frequently occurred. The origin of this early slope angle design is unknown. Today it is the standard practice to design rock slopes to fit the structural geology.

Benches have been designed and incorporated into many high rock slopes, particularly on interstate highways. When the Federal Government embarked on the interstate freeway system, it became apparent that rock cuts much higher than previously employed would be required.

There was no highway design precedent for these high slopes. However, many open pit mines in the United States had reached depths in excess of 600 to 1000 feet (183 to 305 meters). These operations all used benched rock faces for both practical and safety reasons. Even though highway requirements, construction procedures and long-term maintenance considerations were very different than mining requirements, benched slopes on highway cuts became reasonably common, based on mining practice. These benches have resulted in substantial excess rock excavation. Benches that were not cleaned led to the creation of rockfall "launching ramps" so that rocks were projected onto the highway. This lead to the need for excessively wide catch ditches at the toe of the slope.

Highway departments now are paying the price for these early design errors. ***The accepted practice today is not to use slope benches except at soil-rock interfaces.***

8.3. RECOMMENDED DESIGN PRACTICES FOR SAFETY

The sections that follow outline the most effective design and construction techniques to reduce rockfall hazard (Chassie 1992).

8.3.1. Design the Rock Slope Angle to Fit the Predominant Geologic Structure When It Controls Stability

Rock slope angles should be designed to recognize the existing structural geology. Structure refers to joints, bedding planes, foliation, shears, and faults within the rock mass, which are collectively referred to as discontinuities. Geological mapping of rock exposures and/or oriented core drilling is essential to determine the orientation of the discontinuities. Data analysis and interpretation by engineering geologists/geotechnical engineers specifically experienced and trained in this type of work is necessary to determine the "safe" design slope angle.

8.3.2. Avoid Use of Midslope Benches

Use of outdated "template" designs, such as ¼H:1V slopes with 20-foot (6.1 meter) wide permanent midslope benches located every 40 feet (12.2 meters) vertically, should and have been *discontinued* by highway agencies. In the past, the intent of such benches was to "catch" rockfall. However, these midslope benches are usually not accessible for cleanout or, if they are accessible, not regularly maintained. Over time, these benches fill with rock debris. These filled benches then serve as "launching ramps" for continued rockfall and have propelled rockfall horizontally as far as 60 feet (18.3 meters) from the impact point.

Use of benches on these slopes results in major increases in excavation quantity and cost and results in a much higher overall cut slope with negative environmental visual impact. Much more vegetation is removed. The optimum place for a bench is at ditch grade in the form of a catch ditch, where it will effectively contain rockfall and is visible and accessible for maintenance cleanout. The only place slope benches should be permitted is at the soil overburden/top of rock contact.

8.3.3. Require "Controlled" Blasting-Presplitting or Cushion Blasting at the Final Face

Controlled blasting provides the following major benefits versus uncontrolled blasting.

- Greatly reduces blast damage into the slope face.
- Reduces weathering and long term rockfall.
- Reduces the required rockfall catch ditch width.
- Often allows use of a steeper cutslope with resultant lesser excavation volume, right-of-way taken, vegetation removal, and environmental impact.

The standard specifications of most U.S. highway agencies require the use of controlled blasting on rock slopes more than 10 feet (3.1 meters) high and steeper than 3/4:1 (Refer to FHWA publication FHWA-HI-92-001 titled *Rock Blasting and Overbreak Control,* for detailed coverage of rock blasting, including an excellent guide construction specification.)

8.3.4. Use Adequate Width Rockfall Catch Ditches

Even with the use of controlled blasting, not all rockfall can be prevented. Ditches of adequate width and depth will contain the rockfall and keep it off the travelled way.

Where slopes are developed on new construction, or where existing cuts are to be widened and a geotechnical evaluation and/or maintenance history indicate a significant rockfall hazard exists, the above measures should and are being applied by responsible highway agencies.

Vertical slopes are recommended where the structural geology is favorable because rockfall then automatically tends to fall into the ditch. Lesser ditch width can usually be used. Ditch design should consider velocity and bounce height in addition to rock size.

8.4. AESTHETIC CONSIDERATIONS

Common mitigation techniques that can be used in conjunction with the above measures on new slopes or to "retrofit" existing slopes are described in chapter 7.

Where blasting is required to break and excavate the rock, the "natural" rock face appearance desired by some individuals in some agencies is costly and generally impractical to achieve. These faces result in a long-term, ongoing safety hazard and higher maintenance costs.

Most of the travelling public have no objection to uniform slope faces with blast hole fingerprints evident in the slope face. Figures 8-5 and 8-6 show comparisons of controlled and uncontrolled blast results in the same rock cut.

Planting ledges on the slope face also are impractical to construct. These benches fill in with talus and debris that create "launching ramps" for rockfall and are generally not accessible for maintenance. Roots of larger trees that grow into discontinuities act as giant crow-bars during windy conditions, which lever rock from the crest and slope faces. Hence, trees larger than about 4-inch (101.6mm) diameter, which may lever rock, should be removed from steep slope faces and within 6 feet (1.8 meters) of the slope crests. Figures 8-7 to 8-9 show the comparative appearance of an old nonbenched and revegetated slope, the benched slope during reconstruction, and the benched slope after vegetating the benches. The latter is very unattractive and will ultimately lead to rock instability problems.

8.5. HIGHWAY SAFETY POLICY

The current standard of practice is that rock slopes be designed by geological, geotechnical, or rock mechanics specialists. The public's main concerns are that the road be safe, their travel be unimpeded, and their highway tax dollars are spent wisely.

Following a serious boulder accident near Winter Park, Colorado (figure 1-3) in August 1987, where a boulder struck a Graylines tour bus and killed 8 passengers and seriously injured 4 others, the National Transportation Safety Board issued a "Safety Recommendation." This recommendation included reference to a 1985 California

Rockfall Mitigation report, which states, "The State recognizes that the most direct way of minimizing rockfall is to use adequate design criteria and proper construction techniques such as controlled blasting in the design and construction of new slopes...The Safety Board believes that safety benefits would be realized if States adopted a systematic approach similar to the one developed in California for rockfall mitigation."

Figure 8-5. Comparison of noncontrolled blasting (left side of photo) and controlled blasting (right side). Note the potential for ravelling on the left *(Courtesy FHWA)*.

Figure 8-6. Controlled and uncontrolled blasting in a highway cut near Skagway, Alaska. The slope on the right will provide long term stability. Most highway users consider the presplit face to be attractive.

Figure 8-7. Alignment along old I-40 in North Carolina in early 1950. Note the much more pleasing appearance of the revegetated uniform non-benched slope versus the benched slope shown in figure 8-9.

Figure 8-8. Four-lane I-40 under construction. Note multiple benches on the slope. These have lead to rocks bouncing off partially filled benches.

Figure 8-9. Vegetation developed on many bench areas. As the trees get larger, rocks will be levered from the bench crests and could reach the travelled surface. The visual appearance of the isolated vegetation is unattractive *(Courtesy - North Carolina Department of Transportation).*

As a result, the National Transportation Safety Board recommended that the Federal Highway Administration "Issue a Technical Advisory to various States that describes the circumstances of the accident near Winter Park, Colorado on August 10, 1987, encourage the States to use a systematic rock management program, and stress the importance of proper traffic control during maintenance operations." The recommendation was signed by Jim Burnett, Chairman, NTSB, March 24, 1988.

Clearly, ***SAFETY*** is the primary issue in the long term for rockfall control.

8.6. THE ENVIRONMENTAL IMPACT PROCESS

Most States require some form of environmental assessment for new highway construction or reconstruction projects. As a result of this process, outside agencies and the public will have some input on the projects.

The geotechnical or rock slope engineering specialists must inform and advise the agency staff and landscape architects regarding the necessity to use controlled blasting for rock excavation, the need to remove danger trees, and the need to use nonbenched rock slopes except at the contact between soil and rock in the slopes.

Individuals or agencies who wish to mandate or require the imposition of aesthetic criteria for rock slope excavation, *which would result in building a safety hazard*, should be required to assume the long term safety liability (Chassie, 1992).

The States of Colorado and California have established project review groups who meet with the highway engineers and attend public information meetings to give all parties an opportunity to comment on the proposed project and its impact on the environment.

A typical California Environmental Checklist is shown in table 8-1. It will be noted that reference to rock excavation receives minor consideration (point 3).

Table 8-1. Environmental Significance Checklist
California Eagle Falls Rockfall Project

This checklist was used to identify physical, biological, social and economic factors which might be impacted by the proposed project. In many cases, the background studies performed in connection with this project clearly indicate the project will not affect a particular item. A "NO" answer in the first column documents this determination. Where there is a need for clarifying discussion, an asterisk is shown next to the answer.

PHYSICAL. Will the proposal either directly or indirectly:

	Yes or No	If yes, is it Significant? Yes or No
1. Appreciably change the topography or ground surface relief features?		
2. Destroy, cover, or modify any unique geologic or physical features?		
3. Result in unstable earth surfaces or increase the exposure of people or property to geologic or seismic hazards?		
4. Result in or be affected by soil erosion or siltation (whether by water or wind?)		
5. Result in the increased use of fuel or energy in large amounts or in a wasteful manner?		
6. Result in an increase in the rate of use of any natural resource?		
7. Result in the substantial depletion of any nonrenewable resource?		
8. Violate any published Federal, State or local standards pertaining to hazardous waste, solid waste or litter control?		
9. Modify the channel of a river or stream or the bed of the ocean or any bay, inlet or lake?		
10 Encroach upon a floodplain or result in or be affected by floodwaters or tidal waves?		
11. Adversely affect the quantity or quality of surface water, groundwater, or public water supply?		
12. Result in the use of water in large amounts or in a wasteful manner?		
13. Affect wetlands or riparian vegetation?		
14. Violate or be inconsistent with federal, State, or local water quality standards?		
15. Result in changes in air movement, moisture, or temperature, or any climatic conditions?		
16. Result in an increase in air pollutant emissions, adverse effects on or deterioration of ambient air quality?		
17. Result in the creation of objectionable odors?		
18. Violate or be inconsistent with Federal, State, or local air standards or control plans?		
19. Result in an increase in noise levels or vibration for adjoining areas?		
20. Result in any Federal, State, or local noise criteria being equal or exceeded?		
21. Produce new light, glare, or shadows?		

BIOLOGICAL. Will the proposal result in (either directly or indirectly):

	Yes or No	If yes, is it Significant? Yes or No
22. Change in the diversity of species or number of any species of plants (including trees, shrubs, grass, microflora and aquatic plants)?		
23. Reduction of the numbers of or encroachment upon the critical habitat of any unique, threatened or endangered species of plants?		
24. Introduction of new species of plants into an area, or result in a barrier to the normal replenishment of existing species?		
25. Reduction in acreage of any agricultural crop or commercial timber stand, or affect prime, unique, or other farmland of State or local importance?		
26. Removal or deterioration of existing fish or wildlife habitat?		
27. Change in the diversity of species, or numbers of species of animals (birds, land animals including reptiles, fish and shellfish, benthic organisms, insects or microfauna)?		
28. Reduction of the numbers of or encroachment upon the critical habitat of any unique, threatened or endangered species of animals?		
29. Introduction of new species of animals into an area, or result in a barrier to the migration or movement of animals?		

SOCIAL AND ECONOMIC. Will the proposal directly or indirectly:

If yes,is

it

Yes, or

significant?

No | Yes or

No

30. Cause disruption of orderly planned development?
31. Be inconsistent with any elements of adopted community plans, policies or goals, or the California Urban Strategy?
32. Be inconsistent with a Coastal Zone Management Plan?
33. Affect the location, distribution, density, or growth rate of the human population of an area?
34. Affect life-styles, or neighborhood character or stability?
35. Affect minority, elderly, handicapped, transit-dependent, or other specific interest groups?
36. Divide or disrupt an established community?
37. Affect existing housing, require the acquisition of residential improvements or the displacement of people or create a demand for additional housing?
38. Affect employment, industry or commerce, or require the displacement of business or farms?
39. Affect property values or the local tax base?
40. Affect any community facilities (including medical, educational, scientific, recreational, or religious institutions, ceremonial sites or sacred shrines)?
41. Affect public utilities, or police, fire, emergency or other public services?
42. Have substantial impact on existing transportation systems or alter present patterns of circulation or movement of people and/or goods?
43. Generate additional traffic?
44. Affect or be affected by existing parking facilities or result in demand for new parking?
45. Involve a substantial risk of an explosion or the release of hazardous substances in the event of an accident or otherwise adversely affect overall public safety?
46. Result in alterations to waterborne, rail or air traffic?
47. Support large commercial or residential development?
48. Affect a significant archaeological or historic site, structure, object or building?
49. Affect wild or scenic rivers or natural landmarks?
50. Affect any scenic resources or result in the obstruction of any scenic vista or view open to the public, or creation of an aesthetically offensive site open to the public view?
51. Result in Substantial Impacts associated with construction activities (e.g. noise, dust, temporary drainage, traffic detours and temporary access, *etc.*)?
52. Result in the use of any publicly-owned land from a park, recreation area, or wildlife and waterfowl refuge?

MANDATORY FINDINGS OF SIGNIFICANCE

53. Does the project have the potential to substantially degrade the quality of the environment, substantially reduce the habitat of a fish or wildlife species, cause a fish or wildlife population to drop below self-sustaining levels, threaten to eliminate a plant or animal community, reduce the number or restrict the range of a rate or endangered plant or animal or eliminate important examples of the major periods of California history or prehistory?
54. Does the project have the potential to achieve short-term, to the disadvantage of long-term, environmental goals? (A short-term impact on the environment is one which occurs in a relatively brief, definitive period of time while long-term impacts will endure well into the future.)
55. Does the project have environmental effects which are individually limited, but cumulatively considerable? Cumulatively considerable means that the incremental effects of an individual project are considered when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probable future projects. It includes the effects of other projects which interact with this project and, together, are considerable.
56. Does the project have environmental effects which will cause substantial adverse effects on human beings, either directly or indirectly?

The following is a typical project environmental review statement for Installation of Rockfall Protection Measures on Route 101 in the Gaviota Pass Area of Santa Barbara County, California, dated July 5, 1991.

"An initial Study has been prepared by Caltrans. On the basis of this study it is determined that the proposed action would not have a significant effect on the environment for the following reasons:

The project would not significantly affect existing or planned land use, the local economy, or community character. There would be no residential or commercial displacement, growth inducement, or change in traffic patterns.

There would be no significant affect upon historic, cultural or biological resources (including wetlands and endangered species or their habitats). The project would not create significant erosion, seismic hazards, or floodplain encroachment. The project would not significantly affect parklands or scenic resources, nor air, noise, or water quality."

8.7. EXAMPLE AESTHETIC TREATMENTS FOR ROCKFALL MITIGATION

There are aesthetic treatments that can be used on environmentally sensitive areas at reasonable cost. Aesthetic treatments relative to rockfall mitigation projects include use of rock staining, colored or stained concrete or shotcrete, aesthetically treated Jersey barriers, concrete walls and binwalls. They also include use of colored plastic fence and mesh coatings. Where concrete retaining walls or other structures are used pattern designs or special surface finishes may be aesthetically practical at acceptable extra costs (figures 8-10 and 8-11). On rock cuts in environmentally sensitive locations, staining with materials such as Eonite may be effective (figures 8-12 and 8-13).

Where weathered rock is excavated, planting small shrubbery-type vegetation may benefit stability and be practical (figures 8-14). On boulder till slopes, regional compatible vegetation should be introduced to reduce soil erosion from around protruding boulders that may ravel.

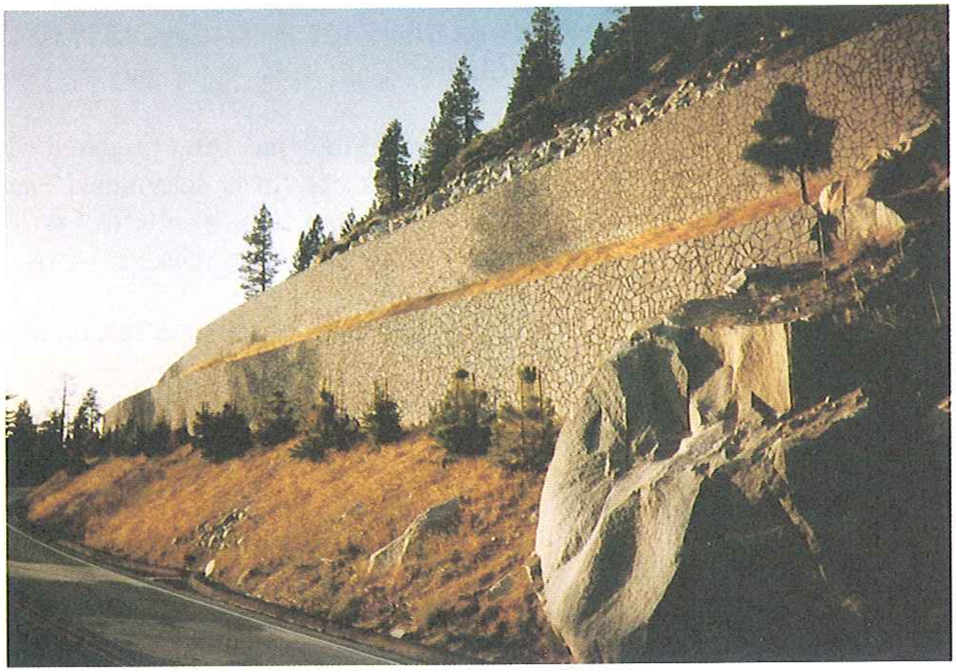


Figure 8-10. Figured facing on concrete retaining walls to improve the appearance. This is a relatively low-cost aesthetic improvement, achieved by the use of commercial form liners *(Courtesy Caltrans)*.

Figure 8-11. Horizontal pattern lines constructed in the bridge pier to match the horizontal bedding in the natural rock. Glenwood Canyon, Colorado *(Courtesy Colorado Department of Highways)*.

Figure 8-12. Rock cut after staining. Christine Falls, Washington *(Courtesy Washington Department of Transportation).*

Figure 8-13. Rock cut before staining *(Courtesy Washington Department of Transportation).*

Figure 8-14. Vegetation planted to improve aesthetics on weathered rock slope *(Courtesy Caltrans).*

Further examples of aesthetic treatments are shown in figures in other chapters.

- Figure 7-16 shows an attractive gabion wall.
- Figure 7-49 illustrates a concrete lock block wall with a granite boulder facing.
- Figure 10-14 shows a catch wall with a very pleasing appearance.
- Figure 10-19 indicates an architecturally developed wall facing.
- Figure 10-24 shows an earth-colored wire mesh draped over the slope.
- Figure 10-23 illustrates a colored Brugg wire rope rock net.

Rock slope and rockfall mitigation designs should be developed by qualified geologists, rock mechanics, or geotechnical engineers who work with road design engineers and not be persons unskilled in rock slope engineering.

CHAPTER 9

SPECIFICATIONS AND CONSTRUCTION

9.1. INTRODUCTION

The majority of rockfall mitigation work is performed on highway projects that were constructed many years ago. As a result, stabilization specifications may be site specific and standard rock slope specifications for new projects may not be appropriate.

For example, the Washington State Department of Transportation treats proposed rock mediation work as unique and highly specialized and requires the development of special contract provisions. This chapter reviews typical provisions.

9.2. PREBID SITE REVIEW

Since the majority of the proposed rock slope remediation involves extensive work on steep rock slopes within a narrow work zone with limited access, a mandatory pre-bid site review is required of each contractor who plans to submit a bid on the proposed project. At the pre-bid site review, all contractors must be accompanied by knowledgeable WSDOT representatives. The bid documents for the project include a certification by a contractor that this requirement has been met, and receipt of this certification is a condition of contract award. All questions and answers relating to the work must be documented and made available to all contractors.

9.3. QUALIFICATIONS OF THE CONTRACTOR

The qualifications and experience of the contractor's work force is the single most important aspect of rock slope remediation work. It is imperative that highly qualified workers at all levels be present on the project to conduct the work in a safe and efficient manner. The special provisions for each major element of the rock slope stabilization work address the minimum qualifications of the workers. In general, supervisory staff (for example, job superintendents and supervisors) are required to have a minimum of five years of demonstrated work experience in rock slope stabilization work, while equipment operators and laborers are required to have a minimum of 2 years. Approval of the contractor's personnel is to be based on resumés that detail each individual's work experience.

9.4. SPECIAL PROVISIONS FOR ROCK SLOPE REMEDIATION WORK

During the PS&E phase of the project, a series of detailed specifications are written that address the construction requirements for post-tensioned rock bolts, rock dowels, rock slope scaling and trimming, horizontal drains, and shotcreting for rock-slope stabilization. These are site specific. Frequently, photographs are taken of each proposed site and the work to be performed is shown on the photographs.

9.5. PROTECTION OF ADJACENT FACILITIES

Any adjacent facilities must be protected as much as reasonably possible from rockfall or blasting damage. As an example, downslope of a project in Chuckanut Drive (SR-11) the "live track" of the Burlington Northern Railroad existed. It was determined that, during the slope scaling and trimming operations, the existing roadway width would not be adequate to catch all of the rock debris removed from the slope. To reduce the possibility of rock debris reaching the railroad, mobile debris deflectors were required below the active work zone.

Prior to train movement below the active work zone, work on the slope was to be suspended and a track patrol would precede the trains to ensure that the track was free of debris.

9.6. CONSTRUCTION CONTRACT TYPE

Under most circumstances, WSDOT contracts for this type of work use a Force Account Contract and pay for the work based on the contractor's daily equipment, labor, and materials used in the progress of the work. This type of contracting requires intensive record keeping and often results in higher costs. To reduce some of the apparent higher costs that are associated with Force Account Contracting, it was recommended that the project be contracted on a Unit Price basis. It was realized that a certain amount of flexibility, in terms of underruns and overruns of each item, would have to be built into the contract since work of this type is difficult to define conclusively until the actual work is in progress. It was proposed that the construction contract be bid with the Increased and Decreased Quantity provision in WSDOT's Standard Specifications waived.

One of the significant issues that is realized during emergency slope stabilization projects is that estimating the actual scope of the work, in terms of definitive for each work element, is very difficult and the quantities for each work element can vary dramatically because of unforeseen site conditions.

9.7. TRAFFIC CONTROL

Traffic control may or may not be supplied by the State. The State has better information regarding the traffic type, volumes, and influence of stoppages than does the contractor and has more experience with this phase of traffic protection. Controls may be provided by flagpersons equipped with signs and radios or by traffic lights.

The following text outlines typical example specifications for rock slope scaling, doweling, bolting, shotcreting, slope drainage meshing and fencing.

9.8. ROCK SLOPE SCALING

9.8.1. Description

This work will consist of removing loose or potentially dangerous blocks of rock on the slope by hand scaling, drag scaling, hydraulic splitters, or blasting (trimming) at locations shown on the plans or as directed by the engineer. The contractor shall supply all materials, equipment and labor required to perform the work specified herein.

9.8.2. Submittals

Not less than two weeks prior to beginning the rock slope scaling, the contractor shall provide to the engineer:

A. Qualifications of the contractor's personnel. The contractor shall provide written evidence that the rock slope scaling foreman and rock slope scalers have performed satisfactory work in similar capacities elsewhere for a sufficient length of time to be fully qualified to perform their duties.

The foreman shall not have less than 5 years of demonstrated experience as a rock slope scaling foreman. The rock slope scalers shall have at least 2 years of demonstrated experience on similar projects.

B. The contractor shall submit a detailed work plan for each rock slope to be scaled. The plan shall detail:

1. The proposed construction sequence and schedule.

2. The types of equipment and tools to be utilized in the work.

3. The number of rock slope scaling crews (crew is defined as one qualified working supervisor, and two qualified scalers) to be employed on the project.

4. Blasting plan for rock blocks requiring light blasting or trimming.

5. Rock removal and disposal plan for rock debris generated from the rock slope scaling work, including provisions to protect adjacent facilities.

6. Traffic interruptions and controls requested.

C. Work shall not begin until the appropriate submittals have been approved in writing by the engineer.

9.8.3. Construction Requirements

The work shall consist of removing loose rock and potentially unstable rock from the rock slopes designated by the engineer. The contractor shall supply all materials, equipment, and labor required for the rock scaling.

Work shall proceed according to the work plan and schedule submitted by the contractor prior to the beginning of the work.

Rock slope scaling shall be conducted on all rock slopes as directed by the engineer and in accordance with the contractor's work plan.

The use of power equipment, such as backhoes, cranes with a drag scaling system, *etc.*, shall be approved by the engineer before use.

The contractor shall provide a qualified rock slope scaling crew that consists of a working foreman and two scalers. The same crew size shall be maintained at all times. Any crew member who must leave for any reason shall be replaced immediately by a qualified replacement. If the scaling activities have the potential of endangering adjacent facilities, the contractor shall provide appropriate protective devices, as per the contractor's work plan, prior to beginning the scaling work.

Rock slope scaling shall start at the top of the slope and work shall proceed downward toward the highway, removing all loose rock blocks as the work progresses. When blasting is required, the explosive force shall be sufficient to remove the rock block but not damage the surrounding rock. If drilling is required as part of the removal process (trimming), the drill holes shall be drilled parallel to the face (straight line) and have a spacing equal to ten times the drill hole diameter. The drill holes shall be loaded with sufficient explosives to break the rock between the drill holes but not damage the new face.

Rock blocks or debris that hang up on the slope during the scaling operations shall be removed upon completion of the first rock slope scaling pass. The new face shall be inspected by the engineer to determine whether or not the rock slope scaling has been completed. If other rock blocks are identified that require removal, the contractor shall continue to scale the slope until the scaling has been completed to the satisfaction of the engineer.

All rock and debris produced during the rock slope scaling operation shall be removed and disposed of by the contractor at locations approved by the engineer.

Traffic control will be provided by the department of transportation (or the contractor).

9.8.4. Measurement

Rock slope scaling will be measured on a crew-hour basis. A crew is defined as a qualified working foreman and two qualified scalers. If power equipment is used, a unit rate for the equipment, operator, and supplies shall be bid at an hourly rate.

9.8.5. Payment

Payment for rock slope scaling will be made at the unit price per crew hour for the item "Rock Slope Scaling." The unit price shall include the cost of furnishing all the materials, equipment, labor, and incidentals necessary to complete the work as specified. Power equipment, other than drills or compressors, shall be paid for on an hourly rate basis.

9.9. ROCK DOWELS TYPE--I (THROUGH THE ROCK)

9.9.1. Description

This work shall consist of the installation of 8-foot and 12-foot (2.4 and 3.7 meters) rock dowels (or other lengths) in accordance with the Standard Specifications, these special provisions, and at location shown on the Plans or as directed by the engineer. The contractor shall supply all materials, equipment, and labor required for the installation of the rock dowels specified herein.

9.9.2. Materials

Materials shall conform to the following:

The rock dowel steel shall conform to ASTM A615 Grade 60 and may be in the form of deformed rebar or the thread bar with a minimum diameter of one (1) inch (No. 8 bar). One end of the rock dowel shall be chamfered to promote easy penetration of resin cartridges. If rebar is used, one end shall be threaded over a length of eight inches. The thread will correspond to the thread of the face plate nuts.

Each rock dowel shall be fitted with a face plate and nut. The face plate shall be of mild steel, not less than ¼ inch (6.35mm) in thickness and not less than 4 inches squared (10.8mm). The plate shall have a central hole large enough to fit easily over the dowel while maximizing the available bearing surface for the washer and the nut. Spherical seating shall not be required.

The nut shall have a minimum dimension across the flat portion of the nut that results in an adequate bearing surface on the washer and face plate. A hardened steel washer shall be placed between the nut and the face plate or the nut and bevelled washer. Bevelled washers shall be used to accommodate nonperpendicular installations.

Epoxy and polyester resin grout shall be proven, nonshrink materials capable of permanently developing the bond and internal strength between the rock dowel and the rock. A single-speed cartridged system shall be used to anchor the dowel in the rock. The cartridge diameter shall be selected in accordance with the recommendations of the manufacturer to ensure complete encapsulation of the rock dowel and satisfactory in-hole mixing. An epoxy or polyester resin shall be selected with a gel time that is consistent with rapid installation. Epoxy or polyester resin to be incorporated into the rock dowel installation shall be within the shelf-life period stated by the manufacturer. Samples of the epoxy or polyester resins shall be provided for testing upon the request of the engineer.

All materials will be approved by the engineer.

9.9.3. Submittals

Not less than two weeks prior to beginning the rock doweling, the contractor shall submit in writing to the engineer for approval:

A. Qualifications of the Contractor's personnel. The supervisors and drill operator shall have a minimum of two years of demonstrated experienced in the installation of rock dowels.

B. The Contractor shall submit a detailed plan for the rock doweling. The plan shall detail:

    1. The proposed construction sequence and schedule.

2. The proposed drilling method and equipment.

3. The proposed drill hole diameter.

4. The proposed steel for the rock dowel including certificates.

5. The proposed bearing plate, flat washer and bevelled washer specifications, including manufacturer's specifications and catalog cuts.

6. The proposed corrosion protection for the rock dowel system.

7. The proposed epoxy or polyester resin specifications including the following:

   a. Gel times and final set times, including details of temperature dependency.

   b. Resin shelf life and batch numbers.

   c. Resin manufacturer's recommendations for mixing times, including temperature dependency.

   d. Resin manufacturer's recommendations for resin storage.

   e. Resin manufacturer's recommended cartridge and hole size for the selected bar diameter.

8. The calibration data for each load cell, test jack, pressure gage and master pressure gage to be used in the proof testing. The calibration tests shall have been performed by an independent testing laboratory within sixty calendar days of the date submitted.

C. Work shall not begin until the appropriate submittals have been approved in writing by the engineer.

9.9.4. Construction Requirements

Work shall proceed according to the work plan and schedule submitted by the Contractor prior to the beginning of the work.

Unless otherwise specified, the angle of installation shall be -15° to the perpendicular to the rock face. The dowels shall be installed within 5 degrees of the specified angle.

Rock dowel steel shall be handled and stored in such a manner as to avoid damage or corrosion. Damage to the rock dowel steel as a result of abrasion, cuts, nicks, welds, and weld splatter will be cause for rejection by the engineer. Rock dowel steel shall be protected from dirt, rust and deleterious substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the engineer will reject the affected rock dowel.

Epoxy or polyester resins shall be stored in accordance with the manufacturer's recommendations.

Prior to installation, all mill scale, flaking rust, and grease shall be removed from the steel. The rock dowel shall be corrosion protected over the entire surface. All exposed parts of the rock dowel, bearing plate, and nut on the surface shall be painted with an approved corrosion protection paint.

The contractor shall drill holes to receive the rock dowel that will be suitable for the particular diameter of rock dowel. The contractor shall flush the drill hole of all drill cuttings and debris with compressed air prior to the installation of the rock dowel. Holes drilled for rock doweling in which dowel installation is considered by the engineer to be impractical, shall be redrilled at the contractor's expense. Sufficient resin cartridges to bond the entire length of the dowel shall be pushed to the back of the hole. The rock dowel shall be pushed into the hole while being steadily rotated by means of a drill or suitable pneumatic and coupling.

After the dowel has been fully inserted, rotation shall continue at the speed and for the duration recommended by the resin manufacturer. The mixing time shall be adjusted for the ambient temperature of the rock mass. The dowel shall be maintained in position until the resin has gelled. When the resin has reached final set, the contractor shall install the face plate washer and nut. The nut shall be torqued to a nominal 100 foot-pounds (13.8km) to ensure proper seating against the rock face. The end of the completed rock dowel shall be trimmed to within three (3) inches (76.2mm) of the rock face.

At the discretion of the engineer, up to five (5) percent, but not less than 3 rock dowels, of the installed rock dowels shall be proof tested. The proof test shall be conducted by the contractor, and the engineer will interpret the results. The rock dowel shall be tensioned to 10,000 pounds (4540kg) with a calibrated hollow-ram hydraulic jack using a bar extension and coupler attached to the rock dowel. Load/extension measurements shall be made during tensioning. The load shall be held for 10 minutes with no loss of load. A rock dowel shall be considered to have failed if any movement of the rock dowel anchorage occurs. The engineer may require additional proof of testing beyond the 5 percent maximum if rock dowels fail the proof testing. All failed rock dowels shall be replaced with an additional rock dowel installed in a separate hole. No payment will be for rock dowels that fail nor for additional proof testing.

9.9.5. Measurement

Rock dowels will be measured for payment at the unit price for each dowel installed and accepted.

9.9.6. Payment

Payment will be made for each of the following bid items:

8-Foot (19.52m) Rock Dowel, per each dowel.
12-Foot (44m) Rock Dowel, per each dowel.
(or other specified length)

The unit contract price for the above listed bid item shall be full pay for furnishing all labor, tools, materials and equipment necessary for the completion of the work as specified.

9.10. ROCK DOWELS TYPE--2 (AT TOE OF BLOCK)

9.10.1. Description

Dowels are used to provide a passive support at the toe of the block that could slide on an adversely oriented surface. Blocks stabilized in this manner are generally smaller than those stabilized with rock bolts, or rock dowels which extend through the unstable block. The contractor shall supply all materials, equipment and labor required for the installation of the rock dowels specified herein.

9.10.2. Construction Requirements

A schematic drawing of dowel support is shown on figure 9-1.

A. The minimum hole depth for the dowel shall be 1 foot (.31 meters).

B. #11 steel reinforcing bar shall be used for the dowel.

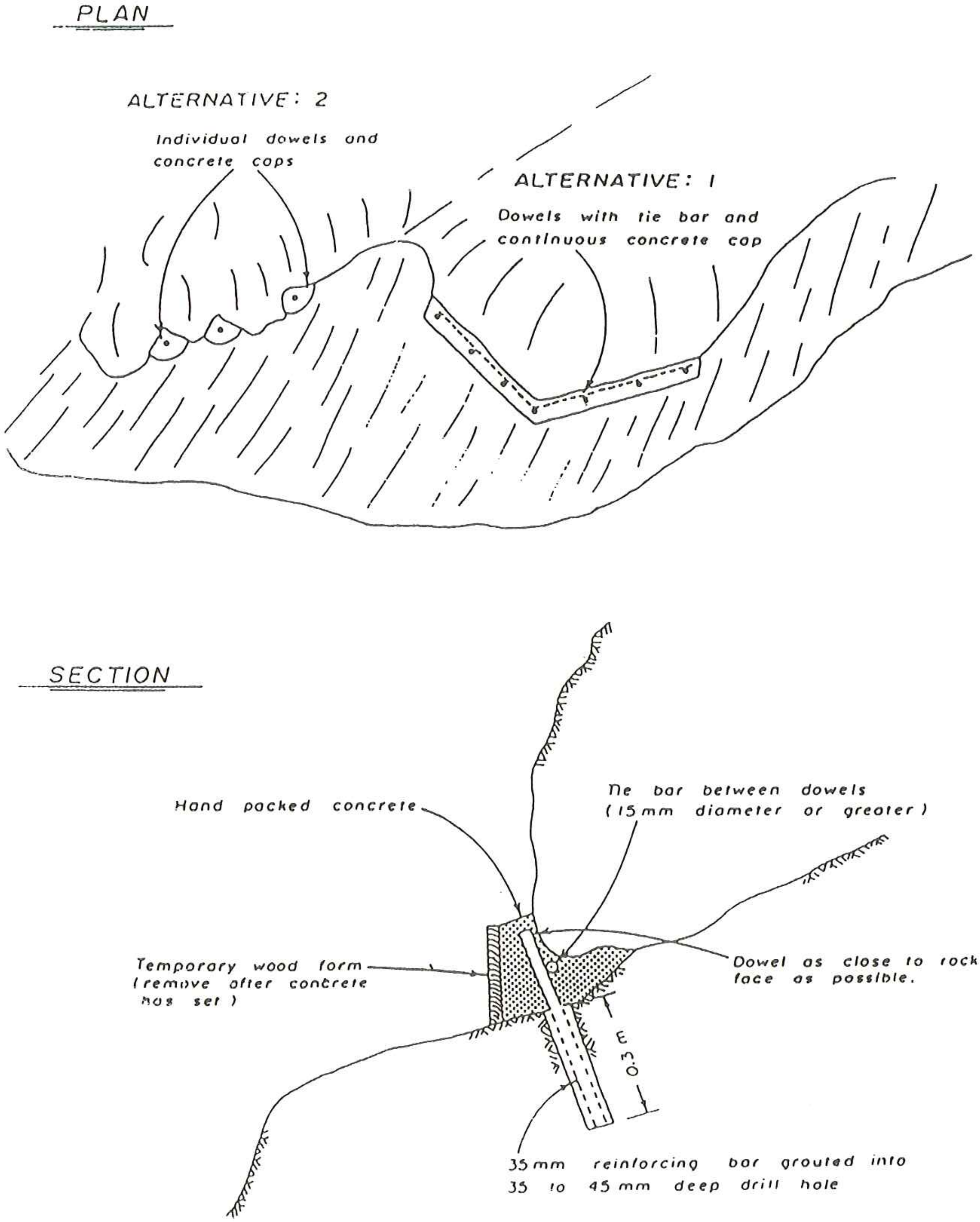
C. The dowel shall be as close as possible to the rock that it is to support.

D. The dowel shall be grouted full depth into the drill hole.

E. The dowel and the toe area of the block that it is supporting shall be encased with shotcrete or hand-packed cement for corrosion protection and support. Wood packing shall not be used.

F. The engineer shall specify the number and spacing of dowels below each rock block.

G. Loose rock above the work area shall be scaled prior to drilling for the dowels to provide a safe work area.



Not to Scale

Figure 9-1. Alternative doweling techniques.

9.10.3. Payment

Payment for rock dowels installed at the toe of an unstable block will be made at the unit price per dowel. The unit price shall include the cost of furnishing all materials, equipment, labor, and incidentals necessary to complete the work as specified.

9.11. ROCK BOLTS

9.11.1. Description

This work shall consist of the design and installation of rock bolts in accordance with the Standard Specifications, these special provisions, and at locations shown on the plans or as directed by the engineer. The engineer shall designate the unbonded length of the rock bolts. The contractor shall select and construct bolts to carry the specified loads and supply all materials, equipment and labor required for the installation of the rock bolts specified herein.

9.11.2. Materials

Materials shall conform to the following requirements:

All rock bolts including anchorages, bearing plates, couplers, corrosion protection, and other appurtenances shall be products of a manufacturer regularly engaged in the manufacturing of rock bolts. Mechanical anchors will not be permitted unless the entire bolt will be grouted after tensioning. Bolts shall be fabricated from deformed bars and be capable of being tensioned.

Grouts, epoxy, or polyester resin shall be proven, nonshrink materials capable of permanently developing the bond and internal strength necessary for tensioning required on the project. If nonshrink cement grout is used, it shall achieve a minimum strength of 6000 psi (41,370kBa) in not more than three days. Fondu cement shall not be used. Epoxy or polyester resins in cartridge form shall be approved by the engineer.

9.11.3. Submittals

Not less than two weeks prior to the beginning of the rock bolting, the contract shall submit in writing to the Engineer for approval:

A. Qualifications of the contractor's personnel. The foreman and the drill operator shall have a minimum of two years of demonstrated experience in the installation of post tensioned rock bolts.

B. The contractor shall submit a detailed plan for the rock bolting. The plan shall detail:

1. The proposed construction sequence and schedule.

2. The proposed drilling methods and equipment.

3. The proposed drill hole diameter and design bond length.

4. The proposed grout mix design, epoxy, or polyester resin specifications, including manufacturers' data sheets and catalog cuts, plus the procedures for placing the grout, epoxy, or polyester resin.

5. The proposed rock bolt couplers, bearing plate, anchor unit, flat washer, and bevelled washer specifications, including manufacturer's data sheets and catalog cuts.

   Anchorage devices shall be capable of developing 95 percent of the minimum guaranteed ultimate tensile strength of the prestressing steel. The anchorage devices shall conform to the static strength requirements of Section 3.1.(1) and Section 3.1.8(1) of the Post-tensioning Institute Guide Specifications for Post-tensioning Materials.

   The bearing plates shall be sized so the bonding stresses in the plate do not exceed the yield strength of the steel when a load equal to 95 percent of the minimum guaranteed ultimate tensile strength is applied.

6. Calibration data for each load cell, test jack pressure gage, and master pressure gage to be used. The calibration tests shall have been performed by an independent testing laboratory and tests shall have been performed within 60 calendar days of the date submitted. The Engineer shall approve or reject the calibrated data after receipt of the data.

7. The proposed stressing procedures and stressing equipment setup.

C. Work shall not begin until the appropriate submittals have been approved in writing by the Engineer.

9.11.4. Construction Requirements

Rock bolt anchorage shall be epoxy, polyester resin, or cement grout. Tension shall be transferred to the rock surface by means of a bearing plate washer and nut.

If the rock face is not close to being perpendicular to the axis of the rock bolt or within the angle provided by the bevelled washer, or the rock under the bearing plate is not sound, a bearing pad approved by the engineer shall be constructed so that the rock bolt is not bent when tension is applied. Where the rock surface is generally weak or weathered, extra large bearing plates, approved by the engineer, shall be used to distribute the load over a larger surface area to reduce the potential of failure of the bearing zone.

The rock bolts shall be handled and stored in such a manner as to avoid damage and corrosion. Damage to the rock bolt as a result of abrasions, cuts, nicks, welds, and weld splatter will be cause for rejection. The rock bolts shall be protected from dirt, rust, and harmful substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the engineer will reject the affected rock bolt.

Prior to installation, all mill scale, flaking rust, and grease shall be removed from the steel. The rock bolt shall be corrosion protected with grout or resin over the entire surface. All exposed parts of the bolt, bearing plate, and nut on the surface shall be painted with an approved corrosion protection paint.

The contractor shall drill holes to receive the rock bolts to the diameter recommended by the rock bolt manufacturer. Where possible, and unless otherwise specified by the Engineer, rock bolts shall be installed in holes drilled 10 to 15° below normal to the rock surface. The contractor shall flush the drill holes of all drill cuttings and debris with compressed air prior to the installation of the rock bolt. Holes drilled for rock bolting, in which bolt installation is considered by the engineer to be impractical, shall be redrilled at the contractor's expense.

The contractor shall select the type of rock bolt and construction method to be used. The rock bolt shall be sized so that the design load does not exceed 60 percent of the minimum guaranteed ultimate tensile strength of the rock bolt. In addition, the rock bolt shall be sized so the maximum test load does not exceed 80 percent of the minimum guaranteed ultimate tensile strength of the bolt.

The contractor shall conduct a performance test to demonstrate the effectiveness of the rock bolt construction method.

Rock bolts shall be tensioned to 120 percent of the design load of the rock bolt. The design load for each rock bolt is 25,000 pounds (11,350kg). The rock bolt shall be tensioned with a calibrated hollow-ram hydraulic jack. Load extension measurements shall be made during tensioning.

The Engineer will analyze the rock bolt test results and determine whether the rock bolt is acceptable. A rock bolt shall be acceptable if:

A. The total elastic movement obtained at the maximum test load does exceed 80 percent of the theoretical elastic elongation of the stressing strength.

B. The rock bolt will carry the maximum test load with a creep rate that does not exceed 0.04 inches (1mm) between one (1) and ten (10) minutes, or 0.8 inches (20mm) per log cycle of time between the ten (10) and sixty (60) minute readings.

Each rock bolt installed shall be proof tested. Proof test shall consist of tensioning the rock bolt to 120 percent of the design load and holding that load for 10 minutes. If no loss of load occurs in this time period, the bolt is accepted.

If the rock bolt fails this proof test, the rock bolt shall be replaced with an additional bolt installed in a separate hole. No payment will be made for rock bolts that fail.

After tensioning, the load shall be locked off at 100 percent of the design load and the remaining portion of the rock bolt grouted with non shrink, non sanded cement grout. Grouting shall be carried out within 3 days of tensioning of the bolt. Grouting must be carried out from the anchor zone to the collar through the hollow core bolt or through a grout tube.

Grouting is required for corrosion protection and to lock the tension stress permanently into the system.

9.11.5. Measurement

Rock bolts will be measured per linear foot of rock bolt installed and accepted.

9.11.6. Payment

The unit contract price per linear foot of "Rock Bolt" shall be full payment to construct the rock bolt as specified.

9.12. SHOTCRETE

9.12.1. Description

This work shall consist of constructing a pneumatically applied shotcrete blanket onto rock/soil surfaces at locations shown on the plans or as directed by the engineer.

These specifications refer to premixed cement and aggregate pneumatically applied by suitable equipment and competent operators.

The shotcrete shall be composed of portland cement, fine and coarse aggregate, and water. Either wet-mix or dry-mix shotcrete may be used. The shotcrete shall be reinforced with either welded wire fabric or steel fibers.

The shotcrete shall be applied according to these specifications and applicable sections of the American Concrete Institute's Guide to Shotcrete (ACI 506R-85).

The contractor shall be responsible for the design of shotcrete mixes and for the quality of shotcrete placed.

9.12.2. Qualifications of Contractor

At least 30 days prior to beginning shotcrete work, the contractor shall provide written evidence that the supervisor, nozzle operator, and delivery equipment operator have performed satisfactory work in similar capacities elsewhere for a sufficient length of time to be fully qualified to perform their duties.

The supervisor shall not have less than 2 years' experience as a shotcrete nozzle operator. The nozzle operator and delivery equipment operator shall have served at least 1 year of apprenticeship on similar applications with the same type of equipment. Prior to the start of shotcreting for this job, nozzle operators shall, in the presence of the engineer, demonstrate their ability to apply shotcrete of the required quality on a test panel. One satisfactory test panel shot in a vertical position for each mix design used during the course of the work shall be the minimum qualification test for nozzle operators before they will be permitted to place shotcrete.

9.12.3. Materials

Materials shall conform to the requirements of the Standard Specifications supplemented and modified as follows:

A. *Prepacked Product*--Premixed and prepackaged concrete product, with or without steel fibers, specifically manufactured as a shotcrete product may be provided for on-site mixed shotcrete, if approved by the engineer. The packages shall contain cement, aggregate and if appropriate, steel fibers conforming to the materials portion of this specification.

B. *Admixtures*--Admixtures shall not be used without permission of the engineer. If admixtures are used to entrain air, reduce water-cement ratio, retard or accelerate setting time, or accelerate the development of strength, they shall be used at the rate specified by the manufacturer and must be compatible with the cement used. Use of calcium chloride accelerating agent will not be permitted. When used, admixtures shall be dissolved in water before introduction into the mixture. Any color additive shall be approved by the engineer before use. Final acceptance will be made following a test section that has been allowed to cure for at least 4 days.

C. *Water*--In addition to the requirements set forth in the Standard Specifications, the water used in the shotcrete mix shall also be free of elements which would cause staining.

D. *Aggregates*--The combined gradation of fine and coarse aggregate used in the shotcrete shall meet the following grading requirements:

Sieve Size	*Percent Passing by Weight*
1/2" (12.7mm)	100
3/8" (9.7mm)	90 to 100
No. 4	70 to 85
No. 8	50 to 70
No. 16	35 to 55
No. 30	20 to 35
No. 50	8 to 20
No. 100	2 to 10

E. *Anchor bars*--Unless shown otherwise on the plans, anchor bars shall consist of No. 5 reinforcement bar bent into an L-shape. The short leg of the L-shaped bar shall be approximately 6 inches (152.4mm) long and the long leg 2 feet (.61 meters) long.

F. *Welded wire fabric reinforcement*--Unless shown otherwise on the plans, welded wire fabric shall be nongalvanized 8 gage with 4 x 4-inch (101.6 x 101.6mm) mesh (101.6 x 101.6mm) (4 x 4 - W2.1 x W2.1) meeting the requirements of AASHTO M 55. The welded wire fabric shall be clean and free from loose mill scale, rust, oil or other coatings interfering with bond.

G. *Steel fiber reinforcement*--When the plans or specifications require the use of steel fiber reinforced shotcrete, the steel fiber reinforcement shall meet the following requirements. Steel fibers shall have a length between 1 and 1 3/8 inches (35.1mm), have blunt or hooked ends, have a length to diameter ratio of less than 80, and shall be cold drawn carbon steel with a minimum tensile strength of 160,000 psi. Only steel fibers manufactured specifically for use in shotcrete applications will be allowed. The steel fiber content shall not be less than 100 pounds (44kg) for each cubic yard of shotcrete. The steel fibers must be premixed with the cement.

9.12.4. Acceptance Sampling and Testing

A. *General*--Shotcrete test panels shall be prepared by the contractor on vertically supported molds. Test panels shall be approximately 24 inches by 24 inches (610mm x 610mm) by a minimum of 3 inches (76.2mm) deep. The material used to form the back and sides of the molds shall be rigid, nonabsorbent and be nonreactive with cement. The shotcrete placement in vertical molds shall be accomplished utilizing the same shotcrete mix, air and water pressure, and nozzle tip as used for the actual placement of shotcrete on production surfaces. The panels shall be left undisturbed and protected at the point of placement for at least 24 hours or until the final set has taken place. The shotcrete shall be applied to a thickness of 3 to 3.25 inches (76.2mm to 89mm), with no sagging.

1. *Preproduction Testing*--The contractor shall prepare at least two test panels for each mix design for testing. The test panels shall be cured using the approved curing compound in a manner similar to the anticipated field

conditions. The engineer shall receive a copy of the mix design and the compressive test results at least 5 days prior to starting any production work. Production shotcrete work shall not begin until satisfactory test results are obtained.

2. *Production Testing*--In the presence of the engineer, the contractor shall prepare at least one test panel daily during shotcrete operations plus one test panel shot whenever the nozzle operator or equipment is changed during the daily work period. The shotcrete panels shall be allowed to cure using the proper curing compound in the field under the same conditions as the production shotcrete.

B. *Compressive strength tests*

1. *Compressive test cylinders*--Compression test cylinders shall be prepared by the contractor by coring 2-inch (50.8mm) outside diameter cores (requires a 2-inch (50.8mm) inside diameter core bit) from the cured shotcrete test panels. Several cores shall be taken from each panel. The cylinders shall be transported to the testing laboratory within three days of being shot in a manner to prevent being damaged.

2. *Shotcrete compressive strength*--The shotcrete shall be capable of attaining 2500 psi compressive strength at 7 days (1800 psi at 3 days) and 4000 psi at 28 days as determined by AASHTO T 22 (ASTM C39-84) testing of compression test cylinders.

NOTE: Higher strength may be required and specified.

C. *Failure of Shotcrete*--Should any shotcrete section be deficient in any of the specified criteria, that section shall be remedied to the engineer's satisfaction at the contractor's expense. Such remedies may include, but not be limited to, removal and replacement of the substandard section.

9.12.5. Equipment

A. *Pump system*--The pump system used to convey premixed shotcrete ingredients shall deliver a uniform and uninterrupted flow of material without segregation or loss of the ingredients. The mixing equipment shall be capable of thoroughly mixing the specified materials in sufficient quantity to maintain continuous placing.

B. *Air compressor*--The air compressor shall be capable of maintaining a supply of clean air adequate for maintaining sufficient nozzle velocity for all parts of the work and for the simultaneous operation of a blow pipe for clearing away rebound. The compressor shall be capable of providing a minimum of 250 cfm per operating nozzle.

C. *Dry-mix process*

1. *Batching and mixing equipment*--The mixing equipment shall be capable of thoroughly mixing the materials in sufficient quantity to maintain continuous application.

2. *Delivery equipment*--The equipment shall be capable of discharging the aggregate-cement mixture into the delivery hose and delivering a continuous stream of uniformly mixed material to the discharge nozzle. The discharge nozzle shall be equipped with a manually operated water injection system (water ring) to direct an even distribution of water through the aggregate-cement mixture. The water valve shall be capable of ready adjustment to vary the quantity of water and shall be convenient to the nozzleman. The water pressure at the discharge nozzle shall be sufficiently greater than the operating air pressure to assure that the water is thoroughly mixed with the other material. The water pressure shall be steady (nonpulsating). Equipment parts, especially the nozzle liner and water ring, shall be regularly inspected and replaced as required.

D. *Wet-mix process*

1. *Batching and mixing equipment*--The mixing equipment shall be capable of thoroughly mixing the specified materials in sufficient quantity to maintain continuous application.

2. *Delivery equipment*--The equipment shall be capable of discharging the premixed materials into the delivery hose and delivering a continuous stream of uniformly mixed material to the discharge nozzle. Recommendations of the equipment manufacturer shall be followed for the type and size of nozzle to be used and for cleaning, inspection, and maintenance of the equipment.

9.12.6. Construction Requirements

A. *Surface preparation*--Immediately prior to shotcrete application, rock surfaces of the areas to be shotcreted shall be scaled of all contaminating and loose material and be thoroughly cleaned by use of air or water jets, or other means approved by the engineer, in order to provide a good bonding surface. Soil surfaces shall be cleaned of loose material by an air jet.

Shotcrete shall not be placed on any surface that is frozen, spongy, or where there is free water. The surface shall be dampened before applying shotcrete.

B. *Shotcrete blanket thickness control*--The thickness of the shotcrete blanket shall be controlled by installing noncorrosive pins, nails, or other gauging devices normal to the face, such that they protrude the required shotcrete thickness outside the face. These pins shall be placed on a maximum 8-foot (2.4 meters)-square pattern. When wire mesh reinforcement is used, a minimum 1-inch (25.4mm) cover of shotcrete shall be placed over the welded wire fabric.

The lower 2 feet (.61 meters) of the rock slope shall not be shotcreted to allow drainage.

C. *Anchor bars*--Unless otherwise shown on the plans, anchor bars shall be placed at approximately 10-foot (3.1 meters) centers maximum, both horizontal and vertical, in 1 1/4-inch (31.8mm) holes drilled into the rock/soil approximately 24 inches deep. The drilled hole shall be blown clear prior to installation of the anchor bar. The drilled hole shall be completely filled with neat cement grout using a grout tube extending to the bottom of the hole. The anchor bar shall be pushed into the grout-filled hole and centered such that the short leg of the L-shaped shaped bar points upward and is located about 1 1/2 inches (38mm) from the rock/soil surface.

Anchor bars shall be installed where soils or weathered rock is to be covered or where wire mesh is to be placed.

D. *Welded wire fabric*--The welded wire fabric shall be installed approximately 1 1/2 inches (38mm) from the rock/soil surface. Sheets of welded wire fabric mesh shall overlap each other sufficiently to maintain a uniform strength and shall be securely fastened at the ends and edges. The edge and end lap shall not be less than 2 meshes in width (approximately 8 inches or 203mm).

E. *Weep holes*--Unless otherwise shown on the plans, weep holes shall be provided throughout the shotcrete mat at 10-foot (31 meters) centers maximum, horizontal and vertical. The weep holes shall be in contact with open points in the natural rock. Prior to shotcreting, survey stakes shall be driven into open joints. Shotcrete shall be applied around the stakes. After the shotcrete has reached the initial set, the stakes shall be removed to leave the drain hole open.

F. *Batching and mixing shotcrete:*

1. *Dry-mix process*--The cement and aggregate shall be batched by weight. Predampening shall be carried out prior to flow into the main hopper and immediately after flow out of the packaging in order to ensure that the premix will flow at a uniform rate (without slugs) through the main hopper, delivery hose and

nozzle to form uniform shotcrete, free of dry pockets. No predampened cement/aggregate mix shall be used if allowed to stand for more than 90 minutes.

2. *Wet-mix process*--Batching and mixing shall be done according to the applicable provisions of ASTM C 94.

G. *Batching and mixing steel fibers*--Steel fibers shall be premixed with the cement prior to batching shotcrete.

H. *Shotcrete application*--Unless shown on other plans, the minimum thickness of shotcrete shall be 2 inches (50.8mm) and the maximum thickness shall be 3 inches (76.2mm) for steel fiber reinforced shotcrete. Where wire mesh is used, the mesh shall be covered with a minimum of 1 inch (25.4mm) of shotcrete.

The shotcrete shall be applied from the lower portion of the area upward so that rebound does not accumulate on the portion of the surface that still has to be covered. Rebound material shall not be worked into the finished product. Rebound is defined as the shotcrete constituents that fail to adhere to the surface to which shotcrete is being applied. It shall not be salvaged and included in later batches. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. When, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until steady flow resumes. A nozzleman's helper, equipped with an air blowout jet, shall attend the nozzleman at all times during the placement of shotcrete to keep the working area free from rebound.

Shooting shall be suspended if:

1. High winds prevent the nozzleman from proper application of the material.

2. The temperature is below 40°F (5°C).

3. External factors, such as rain or seepage, wash cement out of the freshly placed material or cause sloughs in the work.

Construction joints shall be tapered over a minimum distance of 12 inches (305mm) to a thin edge and the surface of such joints shall be thoroughly wetted before any adjacent section of mortar is placed. Square construction joints shall not be permitted.

The surface shall be sounded with a hammer for unsound areas resulting from rebound pockets or lack of bond. Areas, sags, or other defects shall be carefully cut out and replaced with a succeeding layer at the contractor's expense. When fabric reinforcement is used and is damaged or destroyed by such repairs, the damaged area shall be replaced by properly lapped and tied additional wire fabric.

Where a layer of shotcrete is to be covered by a succeeding layer, it shall first be allowed to take its initial set. The initial layer shall be cleaned of all loose material prior to placing succeeding layers.

I. *Finishing*--The shotcrete surface shall be left in the natural gun finish.

J. *Curing*--Air placed shotcrete shall be cured by applying a white pigmented, liquid membrane-forming curing compound, as specified in the Standard Specifications. The curing compound shall be applied immediately after gunning. The air in contact with shotcrete surfaces shall be maintained at temperatures above freezing for a minimum of seven days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials.

9.12.7. Measurement

The area of shotcrete blanket to be paid for will be the number of square feet constructed according to the plans or as directed by the engineer.

9.12.8. Payment

Payment for shotcrete blanket will be made at the unit price per square foot for the item "Shotcrete Rock

Slope Stabilization." The unit price shall include the cost of furnishing all materials, labor, equipment and incidentals necessary to complete the work described in this section.

9.13. BLASTING

9.13.1. Description

Controlled blasting techniques, as covered herein, shall be used for forming highway rock cut slopes at the location shown on the plans or called for in the special provisions.

Controlled blasting refers to the controlled use of explosives and blasting accessories in carefully spaced and aligned drill holes to produce a free surface or shear plane in the rock along the specified excavation backslope. Controlled blasting techniques covered by this specification include presplitting and cushion (trim) blasting.

When presplitting, the detonation of the presplit line shall be *before* the detonation of any production holes. Cushion blasting is similar to presplitting, except that the detonation along the cut face shall be performed *after* the detonation of the production holes. Production blasting, as covered herein, refers to the rock fragmentation blasts resulting from more widely spaced production holes drilled throughout the main excavation area adjacent to the controlled blast line. Production holes shall be detonated in a controlled delay sequence.

The purpose of controlled blasting is to minimize damage to the rock backslope and to help ensure long-term stability. The engineer may require the contractor to use controlled blasting to form the faces of slopes, even if the main excavation can be ripped.

9.13.2. General Requirements

A. *Use of Explosives*--All blasting operations, including the storage and handling of explosives and blasting agents, shall be performed in accordance with the applicable provisions of the Standard Specifications and all other pertinent Federal, State, and local regulations.

Whenever explosives are used, they shall be of such character and in such amount as is permitted by the State and local laws and ordinances and all respective agencies having jurisdiction over them.

The Contractor will conform to all applicable State and Federal laws governing explosives storage. The contractor will submit storage plans along with the type of magazine or explosive storage facility to be used on the job site. The contractor will append to the plan the State or Federal regulations governing explosive storage. The contractor is required to conform to all requirements of State and Federal agencies applicable to explosive storage and will conform to the record keeping, placarding, safe distances and all other requirements concerning storage. Applicable magazine permits will be obtained and displayed as required by State or Federal regulations.

B. *Production Specifications*--The delay elements in blasting caps are known to deteriorate with age. For this reason, it is required that all blasting caps used on the project be one year or less of age.

To ensure the accuracy of firing times of blasting caps, it is required that each cap period come from one lot number. Mixing of lot numbers for any one cap period is prohibited.

Explosives are also known to age and deliver much less than the rated energy. For this reason, it is required that all explosives used on the project be 1 year or less of age. They shall remain in the original packaging.

Bulk explosives, such as ammonium nitrate and fuel oil, may not contain the proper amount of diesel oil, because of evaporation or improper mixing. Low diesel oil drastically reduces the energy content of the explosive and commonly produces reddish brown or yellow fumes upon detonation even in dry blastholes. Product that does not meet manufacturer's specifications will not be used on the project.

When, in the opinion of the engineer, any blasting product is either of excessive age or in what appears to be a deteriorated condition, all work will cease until the product's age or quality can be determined.

No blasting product will be brought to the job site if the date codes are missing. The engineer can require that a product be tested by an independent organization to determine its performance as compared to the manufacturer's data sheet. If product performance or composition deviates by more than 10 percent in any manner from the manufacturer's data sheet, that lot number will be rejected.

9.13.3. Scaling and Stabilization

All rock on the cut face that is loose, hanging, or that creates a potentially dangerous situation shall be removed or stabilized, to the engineer's satisfaction, during or upon completion of the excavation in each lift. Drilling of the next lift will not be allowed until this work has been completed, above the drilling area.

The slopes shall be scaled throughout the span of the contract and at such frequency as required to remove all hazardous loose rock or overhangs.

9.13.4. Production Blasting Operations

A. *Blasting Plan Submittal*--Not less than two weeks prior to commencing drilling and blasting operations, or at any time the contractor proposes to change the drilling and blasting methods, the contractor shall submit a "Blasting Plan" to the engineer for review. The blasting plan shall contain the full details of the drilling and blasting patterns and controls the contractor proposes to use for both the controlled and production blasting. The blasting plan shall contain the following minimum information:

    1. Station limits of proposed shot.

2. Plan and section views of proposed drill pattern, including free face, burden, blasthole spacing, blasthole diameters, blasthole angles, lift height, and subdrill depth.

3. Loading diagram showing type and amount of explosives, primers, initiators, and location and depth of stemming.

4. Initiators sequence of blastholes including delay times and delay system.

5. Manufacturers' data sheets for all explosives, primers and initiators to be employed.

The blasting plan submittal is for quality control and record-keeping purposes. Review of the blasting plan by the Engineer shall not relieve the Contractor of his responsibility for the accuracy and adequacy of the plan when implemented in the field.

When the contract requires the Contractor to retain a blasting consultant to assist with the blast design, all blasting plan submittals must be approved by the blasting consultant.

B. *Production Holes*--All production blasting, including that carried out in conjunction with the blasting test section requirements, shall be performed in accordance with the following general requirements.

Production blastholes shall be drilled on the patterns submitted by the contractor and approved by the engineer. The production blastholes shall be drilled within two (2) blasthole diameters of the staked collar location. If more than 5 percent of the holes are drilled outside of this tolerance, at the option of the engineer, the contractor may be required to refill these holes with crushed stone and redrill them at the proper location.

If the blastholes are plugged or unable to be fully loaded, at the option of the engineer, the contractor may be required to deepen or clean out these holes. The blastholes should all be checked and measured before any explosives are loaded into any of the holes to eliminate any safety hazard resulting from drilling near loaded holes.

All blastholes should reach their desired depth. If more than 5 percent of the holes are short before loading, the contractor may be required by the engineer to redrill the short holes to proper grade at the contractor's expense.

In order to control blasting effects, the contractor must maintain a burden distance that is not more than one half the bench height.

Blastholes will be covered to keep overburden from falling into the holes after drilling.

The row of production blastholes immediately adjacent to the controlled blast line shall be drilled on a plane approximately parallel to the controlled blast line. Production blastholes shall not be drilled closer than 6 feet (1.8 meters) to the controlled blast line, unless approved by the engineer. The bottom of the production holes shall not be lower than the bottom of the controlled blastholes. By approval of the engineer, the bottom of the production hole may be lower than the controlled blastholes by the amount of subdrilling used on the production holes. Production holes shall not exceed 6 inches in diameter, unless approved by the Engineer. Detonation of production holes shall be on a delay sequence toward a free face. Stemming material used in production holes shall be sand or other dry angular granular material, all of which passes a 3/8-inch (9.7mm) sieve.

It is the contractor's responsibility to take all necessary precautions in the production blasting so as to minimize blast damage to the rock backslope.

Payment for production blasting shall be incidental to the contract unit price for roadway excavation.

C. *Blasting Test Section(s)*--Prior to beginning full-scale blasting operations, the Contractor shall demonstrate the adequacy of the proposed blast plan by drilling, blasting, and excavating short test sections, up to 100 feet (30.5 meters) in length, to determine which combination of method, hole spacing, and charge works best. When field conditions warrant, as determined by the engineer, the contractor may be ordered to use test section lengths less than 100 feet.

Unless otherwise allowed by the engineer, the contractor shall begin the controlled blasting tests with the controlled blastholes spaced 30 inches (762mm) apart, then adjust if needed until the engineer approves the spacing to be used for full-scale blasting operations.

Requirements for controlled and production blasting operations covered elsewhere in this specification shall also apply to the blasting carried out in conjunction with the test shots.

The contractor will not be allowed to drill ahead of the test shot area until the test section has been excavated and photographed and the results evaluated by the engineer. If, in the opinion of the engineer, the results of the test shot(s) are unsatisfactory, then, notwithstanding the engineer's prior review of such methods, the contractor shall adopt such revised methods as are necessary to achieve the required results. Unsatisfactory test shot results include an excessive amount of fragmentation beyond the indicated lines and grade, excessive flyrock, or violation of other requirements within these specifications. All costs incurred by the contractor in adopting revised blasting methods necessary to produce an acceptable test shot shall be considered incidental to the contract unit prices for roadway excavation and controlled blasting.

If at any time during the progress of the work, the methods of drilling and blasting do not produce the desired result of a uniform slope and shear face, within the tolerances specified, the contractor will be required to drill, blast and excavate in short sections, not exceeding 100 feet (30.5 meters) in length, until a technique is arrived at that will produce the desired results. Extra cost resulting from this requirement shall be borne by the contractor.

9.13.5. Safety Procedures

A. *Warnings and Signals*--The Contractor will establish a method of warning all employees on the job site of an impending blast. The signal should consist of a 5-minute warning signal to notify all in the area that a blast will be fired within a 5-minute period. A second warning signal will be sounded 1 minute before the blast. An all clear signal will be sounded after the blast so that all in the area understand that all blasting operations are finished.

Five minutes prior to the blast, five long signals on an air horn or siren will be sounded. One minute prior to the blast, five short signals on an air horn or siren will be sounded. The all clear will be one long signal of at least 30 seconds in duration to indicate that all blasting has ceased.

B. *Lightning Protection*--The contractor shall furnish, maintain, and operate lightning detection equipment during the entire period of blasting operations and/or during the periods that explosives are used at the site. Equipment shall be similar or equal to the Thomas Instruments SD250 Storm Alert, as manufactured by DL Thomas Equipment, Keene, New Hampshire. The equipment shall be installed when approved by the engineer. When the lightning detection device indicates a blasting hazard potential, personnel shall be evacuated from all areas where explosives are present. When a lightning detector indicates a blasting hazard, the following shall be performed:

1. Clear the blasting area of all personnel.

2. Notify the project engineer of the potential hazards and precautions to be taken.

3. Terminate the loading of holes and return the unused explosives to the day storage area.

4. If blastholes are loaded and would pose a hazard to traffic if detonated, roads will be closed until the lightning hazard has passed.

5. When the hazard dissipates, inform the project engineer that production blasting will continue.

C. *Check for Misfires*--The contractor shall observe the entire blast area for a minimum of 5 minutes following a blast to guard against rockfall before commencing work in the cut. The 5-minute delay between blasting and allowing anyone but the blaster to enter the area is needed to ensure that no misfires have occurred.

During the 5-minute delay, it is the blaster's responsibility to go into the shot area and check all holes to ensure that they have detonated. If any holes have not fired, these misfires will be handled by the blaster before others enter the work area.

The engineer shall, at all times, have the authority to prohibit or halt the contractor's blasting operations if it is apparent that, through the methods being employed, the required slopes are not being obtained in a stable condition or the safety and convenience of the travelling public is jeopardized.

D. *Misfire Handling Procedures*--Should a visual inspection indicate that complete detonation of all charges did not take place, the following procedures will be followed:

1. If the system was energized and no charges fired for electric systems, the lead wire will be tested for continuity prior to inspection of the remainder of the blast. For nonelectric systems, the lead in or tube will be checked to ensure that detonation has entered the blast area.

2. Should an inspection of the electric trunkline or lead in tubing-line indicate that there is a break in the line or if the tubing did not fire,

then the system will be repaired and the blast refired. If the inspection indicates that the trunkline has fired and misfired charges remain, the blaster will do the following:

a. The blaster will exclude all employees except those necessary to rectify the problem.

b. Nearby roads will be closed if a premature explosion could be a hazard to traffic.

c. The blaster will correct the misfire in a safe manner. If the misfire poses problems that cannot be corrected safely by the blaster, a consultant or an explosive company representative skilled in the art of correcting misfires, will be called to rectify the problem.

9.13.6. Controlled Blasting Methods

A. *Presplitting*--All presplitting, including that carried out in conjunction with the blasting test section requirements, shall be performed in accordance with the following requirements:

- Unless otherwise permitted by the engineer, the contractor shall completely remove all overburden soil and loose or decomposed rock along the top of the excavation for a distance of at least 30 feet (9.2 meters) beyond the end of the production hole drilling limits, or to the end of the cut, before drilling the presplitting holes.

- Potentially dangerous boulders or other materials located beyond the excavation limits shall also be removed as ordered by the engineer. Payment for removal of the material located beyond the excavation limits shall be by force account.

- The presplit drillholes shall not be less than 2.5 inches (63.5mm) and not more than 3 inches (76mm) in diameter.

- The contractor shall control the drilling operations by the use of proper equipment and technique to ensure that no hole shall deviate from the plane of the planned slope by more than 9 inches (228.6mm) either parallel or normal to the slope. Presplit holes exceeding these limits shall not be paid for unless, in the engineer's opinion, satisfactory slopes are being obtained.

- Presplit holes shall be drilled within 3 inches (76mm) of the staked collar location. If more than 5 percent of the presplit holes are outside of the 3-inch (76mm) tolerance, they will be filled with crushed stone, stemmed, and redrilled.

- All drilling equipment used to drill the presplit holes shall have electromechanical or electronic devices affixed to that equipment to accurately determine the angle at which the drill steel enters the rock. Presplit hole drilling will not be permitted if these devices are either missing or inoperative.

- Presplit holes shall extend a minimum of 30 feet (9.2 meters) beyond the limits of the production holes to be detonated or to the end of the cut as applicable.

- The length of presplit holes for any individual lift shall not exceed 30 feet

(9.2 meters) unless the contractor can demonstrate to the engineer that he can stay within the above tolerances and produce a uniform slope. Upon satisfactory demonstration, the length of holes may be increased to a maximum of 60 feet (18.3 meters) upon written approval of the engineer. If greater than 5 percent of the presplit holes are misaligned in any one lift, the contractor shall reduce the height of the lifts until the 9-inch (228.6mm) alignment tolerance is met.

- When the cut height will require more than one lift, a maximum 2-foot (.6 meter) offset between lifts shall be permitted to allow for drill equipment clearances. The contractor shall begin the control blasthole drilling at a point that will allow for necessary offsets and shall adjust, at the start of lower lifts, to compensate for any drift which may have occurred in the upper lifts. Payment for the additional excavation volume, resulting from the allowed 2-foot (.6 meter) offsets, shall be at the contract unit price for roadway excavation.

- Drilling 2 feet (.6 meters) below ditch bottom will be allowed to facilitate removal of the toe berm.

- Before placing charges, the contractor shall determine that the hole is free of obstructions for its entire depth. All necessary precautions shall be exercised so that the placing of the charges will not cause caving of material from the walls of the holes.

- Drillhole conditions may vary from dry to filled with water. The contractor will be required to use whatever type(s) of explosives and/or blasting accessories are necessary toaccomplish the specified results.

- The diameter of explosives used in presplit holes shall not be greater than ½ the diameter of the presplit hole.

- Bulk ammonium nitrate and fuel oil (ANFO) shall not be allowed in the presplit holes.

- Only standard explosives manufactured especially for presplitting shall be used in presplit holes, unless otherwise approved by the engineer.

- If fractional portions of standard explosive cartridges are used, they shall be firmly affixed to the detonating cord in such a manner that the cartridges will neither slip down the detonating cord nor bridge across the hole. Spacing of fractional cartridges along the length of the detonating cord shall not exceed 30 inches (762mm) center to center and shall be adjusted to give the desired results.

- Continuous column cartridge-type of explosives used with detonating cord shall be assembled and affixed to the detonating cord in accordance with the explosive manufacturer's instructions, a copy of which shall be furnished to the engineer.

- The bottom charge of a presplit hole may be larger than the line charges but shall not be large enough to cause overbreak. The top charge of the presplitting hole shall be placed for enough below the collar, and reduced sufficiently, to avoid overbreaking and heaving.

- The upper portion of all presplit holes, from the top charge to the hole collar, shall be stemmed. Stemming materials must be sand or other dry angular granular material and must pass through a 3/8-inch (9.7mm) sieve.

- As long as equally satisfactory presplit slopes are obtained, the contractor, at his option, may either presplit the slope face before drilling for production blasting or may presplit the slope face and production blast at the same time, provided that the presplitting drillholes are fired first. If required to reduce ground vibrations or noise, presplit holes may be delayed, providing the hole to hole delay is no more than 25 milliseconds.

- The presplit slope face shall not deviate more than one foot from a plane passing through adjacent drillholes, except where the character of the rock is such that, as determined by the engineer, irregularities are unavoidable. The 1-foot (.31 meter) tolerance shall be measured perpendicular to the plane of the slope. In no case shall any portion of the slope encroach on the roadbed.

B. *Cushion (Trim) Blasting*--Where the horizontal distance from the cut face to the existing rock face is less than 15 feet (4.6 meters), the contractor may cushion blast in lieu of presplitting. Cushion blasting is similar to presplitting except that the detonation along the cut face occurs *after* the detonation of all production holes. Differences in delay times between the trim line and the nearest production row shall not be greater than 75 milliseconds nor less than 25 milliseconds. With the exception of the above criteria, requirements previously given for presplitting shall also apply to cushion blasting.

C. *Sliver Cuts*--For sliver cuts, pioneering the top of cuts and preparing a working platform to begin the controlled blasting drilling operations may require unusual work methods and use of equipment. The contractor may use angle drilled holes or drilled holes during the initial pioneering operations to obtain the desired rock face. The hole diameter requirements for controlled blasting are applicable for pioneering work. Hole spacing shall not exceed 30 inches (762mm).

9.13.7. Special Requirements

A. *Blasting Consultant*--When called for in the contract special provisions, the contractor shall retain a recognized blasting consultant to assist in the blast design. The blast design shall include both the controlled and production blasting. The consultant shall be an expert in the field of drilling and blasting who derives his primary source of income from providing specialized blasting and/or blasting consulting services. The consultant shall not be an employee of the contractor, explosives manufacturer, or explosives distributor.

Not later than the preconstruction conference, the contractor shall submit a resumé of the credentials of the proposed blasting consultant. The resumé shall include a list of at least five highway rock excavation projects on which the blasting consultant has worked. The list shall contain a description of the projects, details of the blast plans, and modifications made during the project. The list shall also contain the names and telephone numbers of project owners with sufficient knowledge of the projects to verify the submitted information. The blasting consultant must be approved by the engineer prior to the beginning of any drilling and blasting work.

B. *Pre-Blast Condition Survey*--When called for in the contract special provisions, the contractor shall arrange for a preblast surve of any nearby buildings, structures, or utilities which may potentially be at risk from blasting damage. The survey method used shall be acceptable to the contractor's insurance company. The contractor shall be responsible for any damage resulting from blasting. The preblast survey records shall be made available to the engineer for review. Occupants of local buildings shall be notified by the Contractor prior to the beginning of blasting.

C. *Vibration Control and Monitoring*--When blasting near buildings, structure, or utilities that may be subject to damage from blast-induced ground vibrations, the ground vibrations shall be controlled by the use of properly designed delay sequences and allowable charge weights per delay. Allowable charge weights per delay shall be based on vibration levels that will not cause damage. The allowable charge weights per delay shall be established by carrying out trial blasts and measuring vibration levels. The trial blasts shall be carried out in conformance with the blasting test section requirements, modified as required to limit ground vibrations to a level which will not cause damage.

Whenever vibration damage to adjacent structures is possible, the contractor shall monitor each blast with an approved seismograph located, as approved, between the blast area and the closest structure subject to blast damage. The seismograph used shall be capable of recording particle velocity for three mutually perpendicular components of vibration in the range generally found with controlled blasting.

Peak particle velocity of each component shall not be allowed to exceed the safe limits of the nearest structure subject to vibration damage. The contractor shall employ a qualified vibration specialist to establish the safe vibration limits. The vibration specialist shall also interpret the seismograph records to ensure that the seismograph data shall be utilized effectively in the control of the blasting operations with respect to the existing structures. The vibration specialist used shall be subject to the engineer's approval.

Data recorded for each shot shall be furnished to the engineer prior to the next blast and shall include the following information:

1. Identification of instrument used.

2. Name of qualified observer and interpreter.

3. Distance and direction of recording station from blast area.

4. Type of ground at recording station and material on which the instrument is sitting.

5. Maximum particle velocity in each component.

6. A dated and signed copy of seismograph readings record.

D. *Air Blast and Noise Control*--When called for in the contract special provisions, an air blast monitoring system shall be installed between the main blasting area and the nearest structure subject to blast damage or annoyance. The equipment used to make the air blast measurements shall be the type specifically manufactured for that purpose. Peak overpressure shall be held below 0.05 psi at the nearest structure or other designated location. Appropriate blasthole patterns, detonation systems, and stemming shall be used to prevent venting of blasts and to minimize air blast and noise levels produced by the blasting operations. The overpressure limit shall be lowered if it proves too high based on damage or complaints. A permanent, signed and dated record of the peak overpressure measurements shall be furnished to the Engineer immediately after each shot.

E. *Flyrock Control*--Before the firing of any blast in areas where flying rock may result in personal injury or unacceptable damage to property or the work, the rock to be blasted shall be covered with approved blasting mats, soil, or other equally serviceable material, to prevent flyrock.

If flyrock leaves the construction site and lands on private property all blasting operations will cease until a qualified consultant, hired by the contractor, reviews the site and determines the cause and solution to the flyrock problem. Before blasting proceeds, a written report will be submitted to the engineer for approval.

F. *Public Meetings*--The contractor shall make his qualified vibration and air blast specialist and blasting consultant available for one day if requested by the contracting officer to prepare for and participate in a public meeting conducted by the contracting officer to better inform the public about anticipated drilling and blasting operations. The specialists shall be prepared to answer any questions dealing with the magnitude of seismic motion, air blast overpressure, and flyrock expected to impact on the public.

9.13.8. Record Keeping

A. *Daily Explosive Material Consumption*--The contract or shall keep a daily record of transactions to be maintained at each storage magazine. Inventory records shall be updated at the close of every business day. The records shall show the class and quantities received and issued and total remaining on hand at the end of each day. Remaining explosive inventory shall be checked each day and any discrepancies that would indicate a theft or loss of explosive material would be immediately reported.

B. *Report of Loss*--Should a loss or theft of explosives occur, all circumstances and details of the loss or theft will be immediately reported to the nearest office of Alcohol, Tobacco & Firearms, as well as to the local law enforcement authorities and contractor's offices representative.

C. *Daily Blasting Logs*--The contractor shall provide the contracting officer, on a weekly basis, a daily log of blasting operations. The log shall be updated at the close of each business day. The log shall include the number of blasts, times, and dates of blasts. The blasting locations and patterns and all information shown below:

1. Station limits of the shot.

2. Plan and section views of drill pattern, including free face, burden, blasthole spacing, blasthole diameters, blasthole angles, lift height and subdrill depth.

3. Loading diagram showing type and amount of explosive, primers, initiators and location and depth of stemming.

4. Initiators sequence of blastholes including delay times and delay system in each blasthole.

5. Trade names and sizes of all explosives, primers and initiators to be employed.

6. Signature of the blaster in charge.

The blasting logs are for quality control and recordkeeping purposes. Review of the blast log by the Engineer shall not relieve the Contractor of his responsibility for the accuracy and adequacy of the blasting log.

D. *Video Recording of Blasts*--Videotape recordings will be taken of each blast. The tapes or sections of tapes will be indexed in a manner to properly identify each blast. At the option of the engineer, copies of videotapes of blasts will be furnished on a weekly basis.

9.13.9. Measurement

When controlled blasting is specified as a pay item in the bid schedule, measurement shall be per linear foot of controlled blasthole. The lineal feet of controlled blastholes to be paid for shall be the plan length computed from hole collar elevations to a depth of 2 feet (.61 meters) below finished ditch grade. Holes whose misalignment is in excess of 9 inches (228.6mm) shall not be measured for payment.

9.13.10. Payment

The unit contract price per lineal foot of drill hole for controlled blasting shall be full pay for all materials, explosives, labor, tools and equipment needed. Quantities shown in the plans are based on 30-inch (762mm) hole spacing. Actual quantities will depend on field conditions and results from test sections.

9.14. HORIZONTAL DRAINS

9.14.1. Description

This work shall consist of the installation of horizontal drains in existing and/or natural rock slopes, in accordance with the Standard Specifications, these special provisions and at locations shown on the plans, or as directed by the engineer. The contractor shall supply all materials, equipment and labor required for the installation of the horizontal rock drains specified herein.

9.14.2. Submittals

Not less than two weeks prior to starting the installation of horizontal rock drains, the contractor shall submit in writing to the Engineer for approval:

A. Qualifications of the contractor's personnel. The foreman and the drill operator shall have a minimum of two years of demonstrated experience in the installation of horizontal drains.

B. Work shall not begin until the appropriate submittals have been approved in writing by the engineer.

9.14.3. Construction Requirements

The horizontal drains shall consist of a 3-inch (76.2mm) diameter percussion drill hole, drilled at a plus 3° to 5° angle above the horizontal and to a depth as shown on the plans or as directed by the engineer. Each drill hole shall be thoroughly cleaned of drill cuttings with either high pressure air or water.

If the drain hole passes through weak rock or fault zones perforated plastic pipe, such as supplied by Soil Sampling Services, Puyallup, WA, or equivalent, shall be installed full length in the drain hole. The pipe O.D. shall be 2.0 inches (50.8mm).

9.14.4. Measurement

Horizontal drains will be measured by the linear foot of hole drilled.

9.14.5. Payment

The unit contract price per linear foot for horizontal drains shall be full payment for furnishing all labor, tools, materials and equipment necessary for the completion of the work as specified.

Payment shall be for:

A. Unlined drains.

B. Lined drains.

9.15. ROCK PROTECTION FENCES AND SLOPE PROTECTION MAT (CATCH FENCES)

9.15.1. Description

Under this item, the contractor shall furnish and install wire mesh fencing, cables, posts, anchors and tie backs as detailed and at the locations shown on the plans or as directed by the engineer.

9.15.2. Materials

A. *Concrete in Footing and Anchors*--The concrete for footings and anchors shall conform to Class "A" concrete.

B. *Posts and Braces*--Fence posts and braces for the rock protection fence shall be 3-1/2-inch nominal (4-inch O.D.) pipe size, standard weight (Schedule 40), hot-dip galvanized steel pipe conforming to ASTM A 53. The posts shall have a weather tight hot-dip galvanized steel post dome cap securely mounted on the top. Repair all cutting and drilling as well as other damage to the galvanizing.

C. *Cable*--Cable shall be 3/8 inch diameter, 6x19 classification, galvanized wire rope with independent wire rope core made from extra improved plow steel. It shall have a minimum zinc coating of 0.20 oz/ft^2 on all wire, and a minimum breaking strength of 13,000 pounds. Submit an 8-foot long sample of the cable for testing.

D. *Hardware*--All rings shall be drop-forged steel heat treated after forging. Use lightweight wire rope thimbles weighing approximately 13.8 pounds per hundred with the 3/8 inch diameter cable. Galvanize all rings, thimbles, wire rope clips and U-bolts according to AASHTO M 232 (ASTM A 153), Class C, expect casting shall be Class A, and forgings shall be Class B.

E. *Anchor Rods, Guy Anchor Rods and Threaded Rods*--Manufacture all rods from steel meeting the requirements of AASHTO M 183 (ASTM A 36) and galvanize according to AASHTO M 232 (ASTM A 153). Repair any damaged galvanizing according to 02420.10(d).

Anchor rods and threaded rods shall be sized as shown, and be continuously threaded. The length of the rods shall be designated by the engineer for each individual location as dictated by the slope at that location.

F. *U-Bolts*--The U-bolts shall conform to the dimensions shown on the plans, be manufactured from steel meeting the requirements of AASHTO M 183 (ASTM A 36) and be hot-dip galvanized, according to AASHTO M 232 (ASTM A 153) after bending and threading.

G. *Spring Anchorage Assemblies*--Construct spring assemblies at both ends of each run of rock protection fence except for the barrier mounted. The anchorage assembly shall consist of anchor, anchor rod, anchor spring, spring holder, turnbuckle, wire rope clips and wire rope thimble.

Hot-dip galvanize all components of the spring anchorage assembly, except the anchor spring, according to AASHTO M 232 (ASTM A 153). Repair any damage.

- *Concrete Anchors*--Concrete anchors shall be precast or cast-in-place.

- *Anchor Rod*--Size the anchor rod as shown and manufacture from steel meeting the requirements of AASHTO M 183 (ASTM A 36).

- *Anchor Spring*--The anchor spring shall be a helical, flat ended steel spring meeting the requirements of ASTM A 125. The spring shall have a free length of approximately 9 inches with a 1-1/18 inch pitch and shall develop a minimum compressed strength of 6,000 pounds. Furnish a test results certificate according to 00165.60 verifying the anchor spring conforms to ASTM A 125.

- *Spring Holder*--The spring holder shall consist of cast-iron spring washer, 1-inch thick steel plate, four 3/4 inch bolts conforming to ASTM A 307 or SAE Grade 5, and a 3/4 inch eye bolt and bolt turnbuckle with 8 inch take-up all dimensioned and assembled as shown.

- *Wire Rope Clips*--Wire rope clips shall have a 7/6 inch diameter for use with 3/8 inch diameter cable.

- *Thimbles*--Thimbles shall be lightweight wire rope thimbles for use with 3/8 inch diameter cable.

H. *Rock Bolt Post Foundations:*

- *Rock Bolts*--Rock bolts shall be 3/4 inch diameter, continuously threaded, and include the expansion shell anchor complete with a keyhole bearing plate, grout tube, washer and nut. The rock bolts shall meet AASHTO M 31 (ASTM A 615), Grade 70 specifications. All rock bolts shall have rolled threads.

All bolts shall be free of any coating except at the coupling end. The bolts shall be completely fabricated at the point of manufacture under controlled shop conditions.

- *Grout*--Cement used in the grouting of rock bolts shall be Type III portland cement. Ratio of water to cement by weight shall be between 0.38 and 0.50. Add an approved fluidifying agent and commercial grade aluminum powder, or equal, to grout in proportion of 0.005 percent by weight of cement. Before injecting grout, mix mixture for a minimum time of three minutes by means of high-speed mechanical agitator and sieve through a 0.045 inch cloth sieve.

Use grout as soon as possible after thoroughly mixing all ingredients, but in no event more than one hour after addition of water to cement, otherwise it shall be wasted. Use water meeting requirements of Section 02020.

- *Keyhole Plates, Washers and Nuts*--The keyhole plates, washers and nuts shall conform to ASTM F 432. The keyhole plates shall be 3/8 inch flat steel plates providing not less than 6"x6" area for each bolt. Prefabricate each keyhole plate with a 1 inch high, 3/8 inch thick post stabilizing collar. The beveled washers shall be steel or malleable iron. Machine washers shall be hardened steel. All nuts shall be the manufacturer's heavy-hexagonal type. Keyhole plates shall have provision for a grouting tube.

- *Lubricant*--Lubricant for threads shall be molybdenum disulfide grease.

- *Threads of Bolts and Nuts*--Protect the threads of bolts and nuts by a plastic tape or molded protector. Strip off just before installation.

- *Grouting Accessories*--Grout tubes, sealers and other grouting accessories for grouting rock bolts shall be of types recommended by manufacturer and as approved.

- *Tests*--Test according to ASTM F 432 of various the parts that make up a rock bolt.

- *Contractor Furnished Data*--Furnish a test resuls certificate according to 00165.60 verifying conformance of the rock bolts to ASTM F 432.

- *Gabion Wire Mesh Fabric*--Use galvanized steel wire, for gabion wire mesh fabric, meeting the requirements of AASHTO M 279 (ASTMA 641), with a nominal diameter of 0.120 inch, Class 1 coating, and a minimum tensile strength of 60,000 psi. Maximum mesh size shall be approximately 4-3/4 inches with triple twist and hexagonal shape.

- *Hog Ring Fasteners*--Fabricate hog ring fasteners or equivalent from No. 9 gauge, zinc-coated steel wire conforming to AASHTO M 279 (ASTM A 116), Class 1.

9.15.3. Measurement

The quantity to be paid for will be the number of linear feet of fencing furnished and erected.

9.15.4. Payment

The unit price for this item shall include the cost of furnishing all equipment, materials including anchors and tie backs, tools and labor necessary to complete the work.

9.16 CABLE REINFORCED ROCK CATCHMENT FENCE--20 FEET HIGH

9.16.1. Description

Under this item, the contractor shall furnish and install wire mesh rock catching fence and tie-backs as detailed and at the locations shown on the plans or as directed by the engineer.

9.16.2. Materials

Posts shall be W 6 x 8.5 or W 6 x 9 and otherwise conform to the specifications for heavy guide rail posts except that post lengths will be as detailed on the plans. Soil plates will not be required.

Seven-sixteenth (7/16)-inch-diameter (6.5mm) holes will be drilled in one post flange at spacings detailed on the plans to accommodate the wire mesh attachment hardware. One (1)-inch-diameter (25.4mm) hole will be drilled in the web to accommodate three-quarter (3/4)-inch (19mm) cable.

Wire mesh shall be galvanized steel wire conforming to FSS gg-w-461g (Federal Specification), medium hardness, Finish 5 and Class 3 coating. Wire shall be approximately 0.12 inches (11 gage) in diameter with a minimum value of 60,000 pounds per square inch.

Mesh shall be hexagonal woven, triple-twisted steel. Wire mesh shall have a uniform pattern and perimeter edges shall be securely selvaged with wire having at least the same strength as the wire used in the body of the mesh.

The size of the mesh openings shall not exceed four and one half (4 1/2) inches (12.7mm) in the longest dimension. A tolerance of three percent is permitted in the wire mesh dimensions of the manufacturer's stated sizes.

Pipes used for mesh attachments shall be Class A, Schedule 40, one-and-one-quarter (1 1/4) inch-diameter (31.8mm) galvanized pipe.

Where cable is required for fence reinforcement and tiebacks galvanized guide rail cable shall be three-quarter (3/4)-inch diameter (14.3mm) and shall consist of three strands (seven (7) wires per strand) and have a minimum tensile strength of 25,000 pounds (11,350kg) per cable. Guide rail cable and fitting (including steel turnbuckle cable end assemblies, spring cable end assemblies and concrete anchors) shall conform to the requirements of N.Y.S.D.O.T. Standard Sheets 606-1R2 and 606-2R2.

Appurtenances are as follows:

A. U bolts for pipe attachment shall be manufactured of 3/8-inch (12.2mm) stock and have an inside diameter of 1 3/4 inches (44.5mm). U bolts shall be supplied with nuts and washers and galvanized in accordance with ASTM 153.

B. Tire wire shall meet the same specifications as the wire used in the mesh body.

C. Hog Rings shall be galvanized steel, approximately 0.148 inch (3.8mm) (9 Gage) in diameter except that if a locking type ring is used it may be 0.124 inch (3.1mm) (11 Gage) in diameter.

Concrete for fence post footing shall be Class "A" concrete.

9.16.3. Construction Requirements

Galvanized W 6 x 8.5 shall be installed in sixteen-inch diameter holes drilled to a minimum depth of 4 feet 3 inches. Holes shall be backfilled with Class A concrete. Post spacing shall be as indicated on the plans.

Guide rail cable reinforcement, when required, shall be strung through 1-inch diameter (25.4mm) holes in the web of the posts. The cable shall be strung in maximum lengths of two hundred (200) feet (61 meters) and terminated with turnbuckle cable ends at the appropriate posts. The cables shall be tensioned.

The wire mesh shall be secured to the posts using one and one quarter (1 1/4) inch (31.8mm) pipe and U bolts. The mesh shall be tensioned sufficiently to prevent sagging. The mesh shall be secured to the cable reinforcement by either:

A. Continuous weaving with tire wire, or

B. Hog rings on six (6)-inch (152.44mm) centers.

Joints in the mesh shall be overlapped a minimum of 1 foot and fastened as noted above.

TWO LAYERS OF MESH SHALL BE USED.

End posts shall be secured with 3 cables to a concrete anchor block located a minimum of twenty-five (25) feet (7.6 meters) from the end post. Anchor cables shall be secured to the anchor block utilizing a spring cable end assembly and breakaway anchor angle.

Fence posts shall be tied back (at 25 foot or 7.6 meter intervals) to the rock face, utilizing 3/4 inch diameter cables looped around the top and bottom of the posts and secured with wire rope clips. The other end of the cables shall be connected to rock bolts, with turnbuckles, eye bolts and rope clips.

9.16.4. Measurement

The quantity to be paid will be the number of linear feet of fencing furnished and erected.

9.16.5. Payment

The unit price for this item shall include the cost of furnishing all equipment, materials (including terminal anchor blocks and tie-backs), tools, and labor necessary to complete the work.

9.17. BOLTED OR DRAPED WIRE MESH

9.17.1. Description

This work shall consist of installation of wire mesh, designated on the plans, to restrain and channel rockfall. This work shall be done in substantial compliance with the plans, the specifications, the direction of the project Manager and as herein provided. Installation shall be at the locations designated on the plans or established by the project manager.

9.17.2. Materials

A. *Anchor Cable*--Anchor cable shall be 3/4 inch (19.1mm) in diameter, zinc-coated steel wire strand, common grade, Type one coating, conforming with the requirements of ASTM A 475--Zinc Coated Steel Wire Strand.

B. *Resin Grouted Steel Bolts*--Resin grouted steel bolts shall be a minimum of 3/4 inch (19.1mm) in diameter, of the headed type with flash forging. The resin grouted steel bolts shall have the following tensile properties:

Minimum yield strength	43,000 psi
Minimum ultimate strength	70,000 psi

The resin grouted steel bolts shall be manufactured from any of the grades of deformed bars specified ASTM A 615. The resin grouted steel bolts shall be specifically designed for resin grouting with lugs, vertical ribs, and deformations to provide thorough mixing of the resin and to center the resin grouted steel bolts in the drilled holes.

Bearing base plates shall be 6 inches (152.4mm) square or 6-inch-diameter round plates with a thickness that successfully meets the requirements of ASTM F 432.

Resin cartridges shall be of the type used in DU PONT's FASLOC A resin anchored bolts system designed for 0.9-inch-diameter cartridge in a one-inch-diameter (25.4mm) hole with a 3/4 inch (14.3mm) bolt or approved equal.

All materials furnished shall be new and shall be of the best quality and workmanship. The resin grouted bolts system shall be the best standard products of a manufacturer regularly engaged in the production of this type of installation and shall be of the manufacturer's latest approved design.

The contractor shall furnish to the project manager for approval manufacturer's certificates, literature, and shop drawings prior to installation of the units.

C. *Slope Protection Wire and Wire Mesh*

1. *Wire Mesh*--All wire and component steel used in the wire mesh and connections shall conform to all requirements of ASTM A 641 as amended to date, finish five, soft hardness, Class three zinc coating of not less than 0.80 ozs/sq^2 of uncoated wire surface. Uniformity of coating shall equal or exceed 10 1-minute dips by the Preece Test (ASTM A 239).

Certification of resistance to corrosion may be substituted in lieu of requirements for Class three coating as follows: A section of mesh, including twists and/or fastenings forming the mesh, shall be exposed to a salt spray fog test (ASTM B 117) for at least two hundred hours before failure of any part of the mesh. Hard drawn wire conforming to ASTM A 764 for galvanized MB, Type III with the weight of coating being as specified above may be used in lieu of wire of soft hardness as specified above.

2. *Wire*--Wire used in the body of the mesh and lacing wire shall be approximately 0.118 of an inch (3mm) (approximate U.S. gage 11) in diameter, after galvanization.

Ties, clips, hog rings and connecting wire shall be supplied in sufficient quantity for securely fastening all edges of the slope protection wire mesh. Ties, locking clips, hog rings and connectors for fastening selvedged edges shall be nine gage before galvanization. Hard drawn wire clips of approximate wire gage 11-1/2 having a method of closure such that the clip ends interlock firmly may be used in lieu of soft nine gage wire for fastening.

3. *Dimensions*--Slope protection wire mesh shall be supplied as specified in various lengths as shown in the plans. Wire mesh dimensions are subject to a tolerance limit of ± five percent of manufacturer's stated sizes.

4. *Mesh Openings*--Openings of the mesh shall be uniform in size and configured as shown on the plans and shall measure not more than five inches (127mm) in the largest dimension unless shown otherwise on the plans.

5. *Non-ravelling Construction*--The wire mesh shall be fabricated so as to be nonravelling. This is defined as the ability to resist pulling apart at any of the twists or connections forming the mesh when a single wire strand in a section of mesh is cut and the section of

mesh is then subjected to the load test described under Sampling, Testing and Certification, paragraphs 2 and 3.

6. *Mesh Elasticity*--The wire mesh shall have elasticity sufficient to permit elongation of the mesh equivalent to a minimum of five percent of the length of the section of mesh under test without reducing the diameter or tensile strength of individual wire strands to values less than those for similar wire 0.01 inch smaller in diameter.

9.17.3. Sampling, Testing, and Certification

Samples for testing shall include enough samples of each component of the wire mesh protection to complete testing and certification including a proposed mesh jointing construction.

A. *Mesh*--An uncut section of mesh, six feet by three feet (1.8 by .92 meters) minimum, including all selvedge bindings, shall have the ends securely clamped for 3 feet along the width of the sample. When the width of the section under test exceeds 3 feet (1.8 meters) the clamps shall be placed at the center of the width and the excess width shall be allowed to fall free on each side of the clamped section. The sample shall then be subjected to tension sufficient to cause five percent elongation of the sample section between the clamps. Elongation of the mesh under load shall be effected without reducing the diameter or tensile strength of individual wire strands to values less than those for similar wire one gage smaller in diameter.

   After elongation and while clamped as described above (and otherwise unsupported), the section shall be subjected to a load applied to a one square foot (.093m^2) area in the approximate center of the sample section between the clamps and in a direction perpendicular to the direction of the tension force. The sample shall withstand, without rupture of any strand or opening of any mesh fastening, an actual load, so applied, equalling or exceeding 6,000 pounds (2,724kg). The ram head used in the test shall be circular and have its edges bevelled or rounded to prevent cutting of the wire strands.

Wire mesh shall be tested for nonravelling properties by having a single wire strand of mesh cut and then subjecting the section of mesh to the load test described above. The mesh must be able to resist pulling apart at any of the twists or connections forming the mesh or other unravelling under these conditions.

Two sections of mesh 6 feet by three feet (1.8 by .92 meters) minimum with selvedges intact shall be joined in a manner conforming to these provisions and proposed for construction by the contractor shall also be submitted for testing and subjected to the tests described in the preceding paragraphs. Any proposed jointing methods not meeting or exceeding the strength of the mesh body shall not be accepted.

B. *Inspection and Certification*--The contractor shall furnish a certified report of tests, not more than 1 year old, made by an approved testing laboratory showing that the product to be supplied equals or exceeds these specifications.

9.17.4. Construction Requirements

The contractor shall scale slope faces as shown on the plans to remove loose, unstable rock, debris and dispose of said rock and debris at locations approved by the project manager prior to installation of resin grouted steel bolts and draping and anchoring of the slope protection wire mesh or application of shotcrete.

Resin-grouted steel bolt spacing shall be from 3 feet to 6 feet on center or as shown on the plans. During construction, in order to maximize anchorage by selective bolting of large blocks of sound rock or to eliminate large voids between the slope protection wire mesh and the slope face, center to center spacing shall be reduced as directed by the Project Manager.

Rock conditions encountered as construction progresses may require the lengths of the steel bolts to be greater than the minimum 6 foot (1.8 meter) length shown on the plans, and the lengths shall be varied as directed by the project manager.

The varied length shall extend a minimum of one foot (.31 meter) into sound rock.

Where varied lengths are to be utilized, the use of steel bolt couplings (or other approved methods as recommended by the manufacturer) will be permitted.

Where required, resin grouted steel bolts shall be tensioned as recommended by the manufacturer. Minimum torque shall be 200 ft-lbs. After installation, 10 percent of the bolts, randomly selected shall be tested to confirm the specified working load. If a substantial number (one third), of these fail then the contractor shall test all bolts and correct any failing bolts at no additional cost to the Department. When this is completed another set of randomly selected bolts shall be tested and the procedure repeated until satisfactory results are obtained as approved by the project manager.

Boreholes for resin grouted steel bolts shall be drilled approximately perpendicular to the rock face except that it will be permissible, in areas approved by the project manager, to incline the bore holes to a maximum of 30° off the perpendicular in order to intercept and be seated in solid rock.

All drilled holes shall be blown clean with compressed air, minimum of 50 psi, introduced at the back of the hole, upon completion of drilling. The diameter of the drilled holes for the resin grouted steel bolts shall be as recommended by the manufacturer.

The clearance between the steel bolt and bore hole wall shall not be less than 1/8 inch (3.2mm) or more than 3/16 inch (4.9mm).

The manufacturer's recommended installation procedures of resin-grouted steel bolts shall be followed in a careful and diligent manner in order to ensure successful application of the resin bolting.

The slope protection wire mesh shall be lapped a minimum of 6 inches and perimeter edges of wire mesh shall be securely selvedged or bound so that the joints formed by tying the selvedges have minimum strength equal to that of the body of the mesh. Ties, connectors, locking clips or hog rings used for fastening edges shall be spaced four inches apart or less. Perimeter edges may be laced with binding wire by tightly looping through every mesh opening.

Slope protection wire mesh shall extend down the face of the slope and shall be anchored to the face of the slope by resin grouted steel bolts, unless otherwise shown on the plans.

Slope protection wire mesh shall be held in place by the bearing base plates so as to gain maximum stretching and contouring of the existing surface.

In areas where it is determined by the project manager with concurrence of the Materials and Testing Laboratory Bureau representative that bolting the slope protection wire mesh to the rock slope face would be extremely difficult and of doubtful value, the rock bolting will be eliminated and the slope protection wire mesh will be anchored at the top and draped over the rock cut slope face and then anchored at the bottom to contain any loose debris in such a manner that will allow periodical maintenance and debris removal as shown on the plans or by using hooked anchors at the specified spacing and 3/4-inch (19mm) steel cable.

It shall be the contractor's responsibility to maintain the project areas clear of falling rock and debris at all times during construction and at the completion of the project. The contractor shall dispose of waste and debris at locations approved by the project manager.

9.17.5. Measurement

Scaling will be measured by the square yard of slope face. Trenching at the top of the slope, hauling and disposal of rocks and debris shall be considered incidental to the cost of scaling and no measurement will be made therefore.

Slope protection wire mesh will be measured by the square yard complete in place. All ties, lacing wire, clips, hog rings, anchors, plates, or resin-grouted steel bolts will be included in the unit bid price for slope protection wire mesh and no additional measurement will be made therefore.

Steel 3/4-inch-diameter (19mm) anchor cable will be measured by the linear foot complete in place.

9.17.6. Payment

The accepted quantities of scaling will be paid at the unit bid price per square yard of slope face scaled. Trenching at the top of the slope, hauling and disposal of rocks and debris shall be incidental to the cost of scaling and therefore no additional payment will be made.

Slope protection wire mesh will be paid at the unit bid price per square yard complete in place and shall be full compensation for all lies, lacing wire, clips, hog rings, anchors, plates, or resin-grouted steel bolts necessary to complete the work satisfactorily.

Steel 3/4-inch-diameter anchor cable will be paid at the unit bid price per linear foot complete in place.

Payment will be made under:

Payment Item	*Pay Unit*
Scaling of slope face	Square Yard/Meters Square
Slope Protection Wire Mesh	Square Yard/Meters Square
3/4in (19mm) Anchor Cable	Linear Foot

9.18. WIRE ROPE NET ROCK RETAINING SYSTEM

9.18.1. Description

This work shall consist of furnishing, transporting and constructing a wire rope net rock retaining system in compliance with the contract documents and to the lines, grades, dimensions, and at the locations shown on the plans.

The wire rope net rock retaining system shall be as manufactured by Brugg Cable Products, Inc., or L'Entreprise Industrielle or an approved equal. The system shall be designed for the kinetic energy impact loads and height as specified on the plans.

Suppliers can be reached at the following locations:

Brugg Cable Products, Inc.
R.R. 16, Box 197E 11 East Frontage Road
Santa Fe, NM 87505
(505) 438-6161

L'Entreprise Industrielle
Confluence Technologies
15951 Los Gatos Blvd, Suite 15
Los Gatos, CA 95032
(408) 358-4455

It is very important to provide foundation material description, profile, grade, and foundation design.

9.18.2. Materials

All wire rope shall be composed of steel wires that have been individually galvanized prior to being woven into the designated wire rope configuration.

All anchor bolts, nuts, washers and miscellaneous hardware, such as shackles and thimbles, shall be hot-dipped galvanized in accordance with AASHTO M111-80. The fabricator shall grind smooth all welds and rough surfaces prior to galvanizing.

Nets shall be covered with chain link mesh fencing fabric to prevent smaller particles from penetrating the barrier.

Chain link fencing fabric and attaching wire shall be 9 gage, conform to AASHTO M181-86 and shall be zinc coated in accordance with ASTM A392-84, Class 1.

Material for polyvinyl chloride coating, when required, shall conform to the requirements of the Special Provisions for Wire Mesh for Rockfall Control.

9.18.3. Submittals

All submittals in this section shall be approved by the project manager prior to any payment for material on hand as specified in the Standard Specifications.

All submittals in this section shall be submitted to the project manager at least 15 days prior to construction.

The contractor shall submit detailed drawings of the wire rope net rock retaining system, designed for the specified kinetic energy impact loads, to the Project Manager at least 15 days prior to the beginning of construction for review and approval.

The drawings shall include the following minimum information:

Type and diameter of all wire rope;

Braking element locations and number;

Fuse locations and number;

Support posts details;

Anchor locations, type and pull out strength;

Splice details with acceptable locations;

Footing details for different materials;

Concrete type and mix design.

The contractor shall provide the department with a detailed maintenance manual. The manual shall outline the manufacturer's recommendations for servicing and inspection of all components of the system and shall specify the criteria for replacement of all components of the system. The manual shall specify the torque of all bolts and fasteners and the sag limits of all lines in the system. The manual shall also outline recommended procedures for the replacement of net panels, posts, anchors, anchor lines, braking elements and fuses.

The contractor shall submit a certification from the manufacturer that the system is designed to absorb the specified impact loads without passage of the object through the barrier.

The contractor shall submit documentation of test results from field tests of the proposed system. System design shall have been previously field tested and shall have demonstrated satisfactory performance in a similar application and capacity.

9.18.4. Construction Requirements

The wire rope net rock retaining system shall be installed in accordance with the approved submittals, and in the locations shown on the plans or as staked by the project manager.

The contractor shall provide for installation inspection by a qualified manufacturer's representative for all phases of the installation. No separate payment will be made to the contractor for the services of the manufacturer's representative.

The contractor shall test not less than 10 percent of the rock and soil anchors for compliance with the minimum pull-out strength specified. The contractor shall submit a testing plan to the project manager 10 days prior to any testing. The testing method shall be approved by the project manager prior to testing. The project manager shall designate which anchors are to be tested. If a tested anchor fails to meet the specified minimum pull-out strength, the contractor shall test all remaining anchors in the same section of system. All anchors that fail to meet the minimum pull-out strength shall be replaced and tested at the contractor's expense.

9.18.5. Measurement

Wire rope net rock retaining system will be measured by the linear foot. Measurement will be along the ground line of the wire rope retaining system from outside to outside of end posts for each continuous run.

9.18.6. Payment

The accepted quantities of wire rope net rock retaining system will be paid for at the contract unit bid price per linear foot for the height and energy range(s) specified, complete in place, and shall be full compensation for all labor, tools, equipment, materials, testing, chain link fabric and appurtenances necessary for a satisfactory system as described in these provisions and on the plans.

Payment will be made under:

Pay Item	*Pay Unit*
Wire Rope Net Rock Retaining System	Linear Foot

9.19. ROCKFALL FENCE (FLEX POST AND THREADBAR)

9.19.1. Description

This work consists of furnishing and transporting materials and tools to the site, and installing specially designed rockfall fences in accordance with these specifications and in reasonably close conformity with the line, grades and details shown in the plans.

1. Flex Post Fence

2. Threadbar Fence

The section on Rock Fence Foundations and Access contains additional requirements for construction of the Flex Post Fence.

9.19.2. Definitions

A. *Flex Post Fence*--This fence is a division designed rockfall protection device that utilizes the newly developed technology of flexible steel strand fence posts placed in cased holes and connected with mesh and cables as shown on the plans.

B. *Threadbar Fence*--This fence is a designed rockfall and debris protection device that utilizes welded-steel beam fence posts connected by threadbar with attached wire mesh, to divert rockfall and debris away from existing structures.

C. *Post Foundation*--A post foundation is a cased excavation in soil, rock, or a combination of both that has been laterally supported and is ready for installation of the flex post fence post or flex post fence (special) post.

D. *The Environmental Limit or B Line*--is defined as the area within which the contractor shall confine all construction activities, staging and stockpiling of materials and equipment. The Environmental Limit is shown on the plans and cross sections for each location and shall be an area extending no more than 15 feet (4.6 meters) in all directions from the actual fence location as measured along the sloping ground surface. The Designated Access Route shall be considered to be within the usable portion of the Environmental Limits and is shown on the plans and marked in the field. The field markings for the access route shall supersede the access route shown on the plans.

E. *The Maximum Extent of Vegetation Removal or A Line*--is defined as the area within which the contractor shall be allowed to remove the existing vegetation to facilitate installation of the fence and fence foundations. The contractor shall make every reasonable effort to confine his operation to within this area. Stockpiling and storage of materials shall be confined to this area unless otherwise approved by the engineer. The *Maximum Extent of Vegetation Removal* is shown on the plans and cross sections for each location and shall be an area extending no more than 3 feet in all directions from the actual fence location as measured along the sloping ground surface.

F. *The Designated Access Route*--is defined as the access route shown in the plans and is considered to be within the usable area of the Environmental Limits. The B Line for access routes shall be as shown on the plans and as marked in the field. The field markings for the access route shall supersede the access route shown on the plans.

G. *The Post Casing*--is defined as a steel pipe into which the steel prestressing strand for the fence post has been installed and grouted to form the fence post.

H. *A Post Foundation Casing*--is a casing that has been installed into the post foundation excavation.

9.19.3. Materials

The materials to be incorporated in the work shall conform to the following:

A. *Wire Mesh*--Wire mesh shall be of the double twist hexagonal netting type, galvanized in accordance with the ASTM A 153. The wire mesh shall be "Maccaferri Rockfall Protection Netting" or approved equal. All connectors and hardware shall be galvanized in accordance with ASTM A 153.

B. *Prestressing Strand*--Prestressing strand shall be 0.6 inch diameter (15mm), Grade 270 ksi steel, 7 wire type having a center wire enclosed tightly by 6 helical wires and shall conform to ASTM A 416. All strand shall be galvanized in accordance with ASTM A 153 or shall be field coated in accordance with the Standard Specifications and as approved by the engineer.

C. *Galvanized Aircraft Cable*--All cables shall be of the galvanized aircraft cable type and shall be the sizes and dimensions shown in the plans.

D. *Steel Plates*--Steel plates shall conform to ASTM A 36 (AASHTO M183) flat steel or equal and shall be the dimensions and thicknesses shown on the plans.

E. *Binding Wire*--Binding wire shall be American Standard Gage number 9 tie wire and shall be galvanized in accordance with ASTM A 153.

F. *Steel Pipe*--Steel pipe shall be ASA Schedule 40 and shall be of the dimensions shown on the plans. Steel pipe shall be galvanized in accordance with ASTM A 153.

G. *Grout Type A*--The grout used for bonding the strand to the foundation casing shall be rapid setting, nonshrink, hydraulic cement, such as "Fosrock Celtite 10-35 Celroc P," "Sure-Grip Group" or an approved equal. The grout shall be packaged so that it may be mixed easily at the site and shall have flow properties to enable the grout to easily and completely penetrate voids as small as 0.08 inches

(2mm). The grout shall be capable of attaining its initial set within 1 hour and shall have a minimum compressive strength of at least 5000 psi at 48 hours.

H. *Threadbar for Threadbar Fence*--Threadbar shall be of the deformed steel type with continuous threading for the entire length. The threading shall be compatible with the couplings as shown in the plans. The threadbar shall conform to the requirements of AASHTO M31 (ASTM A615), Grade 60 and shall not be epoxy coated. The bars shall be of the sizes and lengths shown on the plans.

I. *Threadbar for Dowels*--Threadbar for rock dowels shall conform to the requirements of the Special Provisions.

J. *Paints*--Paints shall be manufactured and applied in accordance with the Standard Specifications. The paint shall be a color that has been submitted and approved by the Engineer prior to delivery of the material to the site.

K. *Epoxy Coating*--All steel elements designated on the plans as epoxy-coated shall be epoxy-coated under shop conditions and in accordance with AASHTO M 284 for corrosion protection.

L. *Threadbar Couplers*--Couplers for threadbars shall be compatible with the bar size and threading of the bars being used. Couplings shall be capable of developing not less than 125 percent of the specified yield strength of the bar.

M. *Washers*--Washers shall conform to the requirements of AASHTO M293 (ASTM F436) and shall be quenched and tempered to a Rockwell hardness of C38 to C45. The round center hole shall be 1/8 inch (20mm) larger in diameter than the bar to be used. Washers shall be flat or bevelled washers, as shown on the plans, and shall be placed between the plate and the nut. Washers shall be epoxy coated under shop conditions in accordance with AASHTO M284.

N. *Wire Rope Clips*--Wire rope clips shall be compatible with the cable sizes shown in the plans. The bases shall be drop forged carbon steel and the nuts shall be of the heavy-duty hexagonal type. All components shall be galvanized in accordance with ASTM A 153.

O. *Turnbuckles*--Turnbuckles shall be weldless, drop-forged carbon steel and shall be galvanized in accordance with ASTM A 153, and shall be of the size and dimensions shown on the plans.

P. *Carriage Bolts*--Carriage bolts shall be the dimensions shown on the plans and shall conform to ASTM A 307. The carriage bolts shall be galvanized in accordance with ASTM A 153.

Q. *Hexagon Head Bolts*--Hexagonal head bolts shall be the dimensions shown on the plans and shall conform to ASTM A 307. The hexagonal head bolts shall be galvanized in accordance with ASTM A 153.

R. *End Caps for Steel Pipes*--End caps for steel pipes shall be compatible with the dimensions and threading of the steel pipes shown in the plans. End caps shall be galvanized in accordance with ASTM A 153.

9.19.4. Construction Requirements

An adequate supply of rock fence materials shall be maintained at the site to prevent delay in the work.

A. The locations, lengths, number of posts, and post locations shown in the plans are approximate. Conditions encountered during the work may indicate that these characteristics, as shown in the plans, should be varied. The engineer may increase, delete, or otherwise alter the locations, lengths, quantities or other characteristics as necessary.

B. To ensure safety of personnel and construction operations during the work the contractor shall bring to the attention of the engineer any location of the rock slope that he feels may constitute a potential hazard. With the approval of the engineer, the area shall be scaled or reinforced as necessary.

Scaling shall be considered incidental to the work and shall be done at no additional cost. Rock reinforcement shall be paid for at the contract unit price in accordance with the Special Provisions.

C. With the approval of the engineer, the contractor may scale small areas of rock to facilitate installation of the fences. Such scaling shall be considered incidental to the work and shall be completed at no additional cost.

D. Safety of the work shall be the responsibility of the contractor. The work shall be performed in a manner to minimize hazards and exposure to the public, construction personnel, and equipment to hazardous and potentially hazardous conditions. The work shall be scheduled so as to ensure safety and to be commensurate with provisions of the Special Provisions.

E. Steel elements that are designed in the plans as painted shall be given one coat of primer and one coat of paint in conformity with the Standard Specifications. Paint shall be manufactured to meet Fed. Spec. TT-E-529G and the color shall be as approved by the engineer. All exposed portions of galvanized steel shall conform to the following:

1. All foreign substances shall be removed.

2. One coat of bonding agent shall be applied uniformly on the surface. Bonding agent shall be Vinyl Butyral Wash Primer conforming to MIL-P-15328 (SSPC Paint No. 57). After the elastomeric Bonding agent has dried sufficiently (<u>not more than 24 hours</u>), one coat of exterior enamel shall be applied by a suitable method that will ensure a uniform coating free of blemishes, discontinuities, bare edges, or other imperfections. Defective surfaces shall be touched up with paint as required.

Painting will not be paid for separately but shall be included in the work.

9.19.5. Shop Drawings

The contractor shall furnish shop drawings at least two weeks prior to beginning the work. Shop drawings shall include the following:

A. A brief narrative describing the contractor's intended procedure for transporting the materials and tools to the fence locations and proposed methods for constructing the fences.

B. The contractor shall submit a color sample and all technical literature for the paint to be used for field coating of the fences.

9.19.6. Certificates

The following certifications shall be submitted to the engineer at least two weeks prior to beginning the work:

A. Certificates of compliance attesting proof of compliance with the specifications.

Certificates are required for the following:

1. Threadbar, including mill reports indicating tensile yield point and elongation results.

2. End hardware.

3. Prestressing strand.

4. Cables.

B. A copy of the appropriate Welders' Certificate issued by the American Welding Society for each individual who will perform welding duties on the project.

9.19.7. Construction Requirements--Flex Post Fence

A. *Installation of fence post in the foundation casing*--The contractor shall install all strands of the fence post into the foundation casing as shown on the plans. A clamp may be used to facilitate installation of the strands into the foundation casing. All strands shall extend 3 feet (9 meters) into the foundation casing with a tolerance of plus or minus 2 inches (50.8mm). There shall be not less than 16 inches (406mm) and not more than 20 inches (508mm) of uncased strand between the top of the foundation casing and the bottom of the fence post casing as shown on the plans. The fence posts shall be installed vertically and shall not vary from vertical by more than 2 percent of the total length of the fence post.

B. *Cross cables*--Cross cables with a nominal diameter of 5/16 inch shall be installed so that the cable does not sag more than 2 inches (50.8) at any point between the posts.

Cross cables with a nominal diameter of 7/16 inch (12mm) shall be tightened at the turnbuckle as shown in the plans. The 7/16-inch (12mm) cables shall not have more than 1/2 inch (12.7mm) of sag between the fence posts.

C. *Turnbuckles*--Turnbuckles shall be placed every 150 feet (45.8 meters) as shown on the plans to achieve the specified taughtness in the cable. The cost of the turnbuckles and installation shall not be paid for separately but shall be included in the bid item price.

D. *Bolts*--Bolts shall be installed as shown on the plans and shall be torqued to not less than 15 ft-lbs and not more than 25 ft-lbs.

E. *Wire rope clips*--The wire rope clips shall be placed on the cables in the configuration shown on the plans or as recommended by the manufacturer. The clips shall be tightened to the following torques:

1/4 inch (6.4mm) cable	15 ft-lbs.
5/16 inch (7.9mm) cable	25 ft-lbs.
7/16 inch (11mm) cable	50 ft-lbs.

F. *Wire mesh and cable*--The wire mesh and cable shall be placed on the downhill side of the posts, as shown in the plans. Two layers of wire mesh shall be placed only on the lower portion of the fence as shown in the plans.

9.19.8. Construction Requirements--Threadbar Fence

A. *Installation of fence posts*--The fence posts shall be placed vertically and shall not vary from vertical by more than 2 percent of the total length of the post.

The flange holes for the threadbar shall be located in the west half of the flange for all post locations.

Levelling pads for the posts shall be required as directed by the engineer to provide a smooth and level platform for the flange plate. The grout used for the levelling pads shall conform to the requirements of the Special Provisions.

1. The threadbar within the fence shall be tightened so that there is no more than 1 inch of sag in the bar at any point between the posts.

B. *Rock dowels*--The rock dowels for the post anchors shall be installed in accordance with the Special Provisions.

9.19.9. Field Coatings on Fences

A. Where shown in the plans, the fences and associated hardware shall be painted in accordance with the Standard Specifications. All surfaces to be painted shall be prepared in accordance with the paint manufacturer's recommended procedure and shall be free of all rust, dirt, grease, or other deleterious matter that might impair the ability of the coating to adhere to the surfaces being painted. The paint shall be a color that has been submitted and approved by the engineer.

B. The contractor shall take precautions to prevent damage, caused by his painting operation, to the area surrounding the fence. Areas that have been inadvertently painted shall be restored to their original condition, as determined by the engineer, at no additional cost.

9.19.10. Measurement

A. Flex Post Fences shall be measured by the number of linear feet of fence installed and accepted. The fences shall be measured for payment as shown on the plans. Foundation will be paid for separately.

B. Threadbar Fence shall be measured by the number of linear feet of fence installed and accepted. Foundation anchors for these fence types shall not be paid for separately but shall be included in the bid item price. Fence post anchors and threadbar fence foundation anchors shall not be paid for separately but shall be included in the bid item price.

9.19.11. Payment

Pay Item	*Pay Unit*
Flex Post Fence	Linear Foot
Threadbar Fence	Linear Foot

Payment will constitute full compensation for furnishing, fabricating, and installing all components of each fence type, except for cased fence foundation holes and access, furnishing and transporting to the work site all materials, tools, labor, equipment, and incidentals necessary to complete the designated pay items in accordance with the Standard Specifications.

Additional fences or additional fence quantity directed by the engineer will be paid for at the contract unit prices, except where ordered to replace unacceptable installations.

Upon completion of the work, all fence materials on hand will remain the property of the contractor. The costs of materials that are to remain the property of the contractor that have been paid to the contractor as materials on hand shall be deducted from other monies due to the contractor.

9.20. ROCK FENCE FOUNDATIONS AND ACCESS

9.20.1. Description

This work consists of furnishing materials, labor, and tools; developing an access route; transporting materials, labor, and tools to the site; consolidation grouting of materials around foundation locations; drilling or hand excavating post holes; installing and stabilizing steel casing in post holes; and cleaning and preparing the post holes for installation of rockfall fences.

This section shall apply only to fence post foundations for the Flex Post Fence.

Development of an access route to each fence site, and daily access for materials, labor, tools, and equipment will not be paid for separately but shall be considered incidental to the work. A specific access route has been identified in the plans for each work site. The contractor shall be constrained to using these routes for access and egress of the work sites.

In order to meet environmental requirements the contractor shall develop access routes in such a way as to minimize the visual impact created by development of the routes. Minor amounts of clearing and grubbing, small amounts of earthwork using hand held tools and establishing safety lines and equipment will be necessary for most of the routes.

The following three fence post foundation types have been established based on assumed ground conditions:

1. Post Foundation in Rock (bid item Rock Anchors).

2. Post Foundation in Talus (bid item Rock Tiedown Anchor (cement grouted).

3. Post Foundation in Soil (bid item Concrete Foundation Pad).

9.20.2. Definitions

A. *A Post Foundation in Rock* is defined as a post hole foundation in which foundation casing has been installed according to this Special Provision. A Post Foundation in Rock shall have been drilled with hand-held or other drilling equipment in rock for the full depth and diameter of the hole. This includes foundations in bedrock or foundations in boulders large enough to fully encompass the post hole for its entire depth. The bid item associated with this foundation type is Rock Anchor.

B. *A Post Foundation in Soil* is defined as a post hole foundation in which foundation casing has been installed and stabilized with Type A grout in accordance with this Special Provision. A Post Foundation in Soil shall have been excavated using hand-held tools in soil and small rock up to 6 inches (152mm) in diameter, and generally free of voids or openwork gravels or cobbles. The bid item associated with this foundation type is Concrete Foundation Pad.

C. *A Post Foundation in Talus* is defined as a post hole foundation in which foundation casing has been installed and stabilized with Type A grout in accordance with this Special Provision. A Post Foundation in Talus shall have been excavated using hand-held tools or drilled through consolidation grouted natural materials consisting of soil, cobbles and boulders with voids. The bid item associated with this foundation type is Rock Tiedown Anchor (Cement Grouted).

D. *A Stabilized Foundation Casing* is one which has been confined with grout or other means in accordance with this Special Provision to prevent horizontal and vertical movements within the hole.

E. *A Post Foundation Casing* is defined as a steel pipe installed in the post foundation excavation.

9.20.3. Materials

The materials to be incorporated in the work shall conform to the following:

A. *Grout Type A*--The grout used for stabilizing the post foundation casing in the foundation excavation shall be rapid setting, nonshrink hydraulic cement and superplasticizer such as "Fosroc Celtite 10-35 Celroc P" or approved equal. The grout shall be packaged so that it may be mixed easily at the site and shall have flow properties to enable the grout to easily and completely penetrate voids as small as 2 millimeters (.078in). The grout shall be capable of attaining its initial set within 1 hour and shall have a minimum compressive strength of at least 5000 psi at 48 hours.

B. *Post Foundation Casing*--The steel casing shall be ASA schedule 40 and shall be of the dimensions shown on the plans. The casings shall be galvanized in accordance with ASTM A 153.

9.20.4. Shop Drawings

The contractor shall furnish shop drawings as specified in this Special Provision. Shop drawings shall include the following:

A. A brief narrative describing the contractor's proposed method of performing the work for each post foundation type, including the proposed method, materials, and equipment to be used for excavating the post holes and delivering labor, materials, and equipment to the work site. The narrative shall discuss the contractor's proposed method of delineating of the Environmental Limits (B Line) and the Maximum Extent of Vegetation Removal (A Line) in the field.

B. The contractor's proposed mix design for Type A grout, and the contractor's proposed methods and equipment for mixing the grout at the work site.

C. A narrative describing the amount and general nature of the work required for development and maintenance of a safe and suitable access route at each location. The submittal shall discuss the contractor's proposed method of reclaiming the access route.

9.20.5. Construction Requirements

A. An adequate supply of post foundation materials and equipment shall be maintained on the site to prevent delay in the work.

B. The quantity, locations, foundation type, and depths of the post foundations shown in the plans are approximate. Conditions encountered during the work may indicate that these items as shown in the plans should be varied. The engineer may increase, delete, change location, type or depth, or otherwise alter the post foundations as necessary.

C. To ensure the safety of personnel and construction operations during the work, the contractor shall bring to the attention of the engineer any location on the rock slope that he feels may constitute a potential hazard. With the approval of the engineer the area shall be scaled or reinforced as necessary.

Scaling shall be considered incidental to the work and shall be done at no additional cost to the division. Rock reinforcement shall be paid for at the contract unit price in accordance with the Special Provisions.

D. With the approval of the engineer, the contractor may scale or move small areas of rock to facilitate construction of the post foundations. Such work shall be considered incidental to the work and shall be done at no additional cost.

E. Safety of the work shall be the responsibility of the contractor. The work shall be performed in a manner to minimize hazards and exposure of the public, construction personnel, and equipment to hazardous and potentially hazardous conditions. The work shall be scheduled so as to ensure safety and to be commensurate with the Commencement and Completion of Work provisions of the Special Provisions.

F. The contractor shall be responsible for developing and maintaining a safe and suitable access route to the work sites. The access routes shall be developed within the Environmental Limits or "B Line," as shown on the plans and as marked in the field by the Division. The contractor shall reclaim, to the satisfaction of the engineer, those areas disturbed by development of the access route. Maintenance, development, and reclamation of the access routes shall be considered incidental to the work and shall be done at no additional cost.

G. The department will provide clear demarcation of the Environmental Limits and the Maximum Extent of Vegetation Removal in the field at each fence location prior to the contractor beginning any work on the fence or the fence foundations.

H. The contractor shall conduct all portions of his operation within the area defined as the Environmental Limit or "B Line." His operations shall be planned and conducted in such a way as to minimize disturbance of the existing vegetation and natural features. The contractor shall not encroach beyond the Environmental Limit with any equipment, materials or personnel for any reason whatsoever without the prior approval of the engineer. Violation of this requirement shall be considered due cause to assess liquidated damages as defined in the Special Provisions.

I. The contractor shall not remove any existing vegetation outside of the area defined as the Maximum Extent of Vegetation Removal or "A Line." Violation of this requirement shall be considered due cause to assess liquidated damages as defined under the Special Provisions.

All waste materials generated by removal of existing vegetation within the "A Line" shall be disposed of by the contractor in a manner acceptable to the engineer at no additional cost.

9.20.6. Construction Details

A. The contractor shall excavate or drill the post foundation holes by whatever means necessary to the full depths and at the locations shown in the plans or as directed by the engineer. Any excess materials or spoils from the excavation shall be scattered or disposed of in a manner acceptable to the Engineer at no additional cost.

B. The correct size and length of post foundation casing shall be installed in the excavations as shown in the plans. The casing shall fit snugly into the hole or be grouted into the hole with Type A grout. The annulus surrounding the outside of the casing shall not be larger than 1/16 inches (1.6mm) at any point on the circumference of the casing for grouted casings and not larger than 1/8 inches (3.2mm) for casings in rock. The casing shall be installed vertically or as directed by the engineer.

C. The post foundation hole shall be cleaned of all soil, rock and debris for the full depth of the casing and shall be plugged to prevent infiltration of water and materials into the casing.

9.20.7. Measurement

Post Foundations shall be measured per complete and accepted foundation for the appropriate bid items listed below. The engineer shall determine which bid item shall apply for each foundation.

9.20.8. Payment

The accepted quantities of work will be paid for at the contract price per unit of measurement for the pay items listed below:

Pay Item	*Pay Unit*
211 Rock Anchor	Each
211 Rock Tiedown Anchor (Cement Grouted)	Each
613 Concrete Foundation Pad	Each

Payment will constitute full compensation for furnishing, fabricating and installing all components of the foundations; drilling, or excavation and disposal of excess materials generated from the excavations; furnishing and training personnel in the use of safety equipment and techniques; developing, maintaining and reclaiming the designated access routes; and furnishing and transporting to the site all materials, labor, equipment and incidentals necessary to complete the designated pay items in accordance with the Standard Specifications.

Additional post foundations of any type, or changes in foundation types or quantities that are directed by the engineer shall be paid for at the contract unit prices, except where ordered to replace unacceptable installations.

Upon completion of the work, all foundation materials on hand will remain the property of the contractor. The costs of materials that are to remain the property of the contractor and that have been paid to the contractor as materials on hand shall be deducted from other monies due to the contractor.

CHAPTER 10

CASE EXAMPLES

10.1. INTRODUCTION

To illustrate a range of practical rockfall problems, evaluations that were required to assess the problem or design and construction procedures used to mitigate the problem, 12 case examples from around the country have been documented and are summarized. The examples selected illustrate the variety of conditions, problems, design considerations, site access difficulties, and mitigation procedures that exist.

A review of the case examples illustrates the need to understand the structural geology, evaluate the rockfall potential at locations well above the highway, and to consider the implications on traffic movement during the stabilization program. The case examples indicate the importance of developing a proactive policy rather than reactive policy where the results of rockfall--sometimes deadly--must be confronted.

10.2. VALDEZ, ALASKA

In the spring of 1991, an unloaded flat deck truck travelling about 14 miles (22.5km) north of Valdez, Alaska ran into a rockfall that covered the highway (figure 10-1). Although the driver slammed on his brakes, the truck ran up and over part of the fall. Fortunately, a metal guardrail kept the truck from diverting into the river. The impact tore the front wheels and undercarriage from the truck (figure 10-2), and the driver received only bruises to his chest, shoulders and head.

The trucking company launched a lawsuit against the State for injury and damages for about $80,000. The writer was retained to review the accident and geotechnical conditions and provide a report.

Site Conditions--The horizontal sight distance was about 250 feet (76.3 meters). With skid marks extending almost from the curve, it appears the driver was exceeding the 45-mi/h (30km/h) speed limit. The asphalt was dry.

Figure 10-1. Empty flat-bed truck that ran into a rockfall of about 30 yd^3. Note the long skid marks. The metal guard rail prevented the truck from being diverted into the river *(Courtesy Alaska Department of Transportation)*.

Figure 10-2. The front wheels and undercarriage were torn off the truck during the accident. The driver (right) suffered moderate bruises *(Courtesy Alaska Department of Transportation)*.

The roadway comprised two, 12-foot (3.7 meters) wide traffic lanes with 8-foot (24 meters) paved outer shoulders. A flat ditch 12 to 15 feet (3.7 to 4.6 meters) wide existed along the inner shoulder. A natural vertical rock face about 20 feet (6 meters) high extended above the ditch. For a further 15 feet, the rock face sloped at about 45° with a rock overhang about 6 feet (1.8 meters) wide and 20 feet (6 meters) high. Beyond the crest, gently sloping rock existed. No previous rock excavation had occurred at the accident site.

The hard metamorphosed rock had two sets of joints that dipped about 60° toward the road to form a large wedge. A third set of joints dipped into the slope at about 30°. A volume of about 30 yd^3 ($23m^3$) fell, bounced on the upper slope, likely partially broke up, fell into the ditch and shoulder, further breaking up and rolled and bounced into the traffic lanes. The ditch capacity would have been inadequate to retain all the rockfall.

Some water was flowing over the face and from some of the joints. Small trees existed near the crest of the slope. There was no record of past rockfall at this site.

The Valdez area had many small earthquakes each year, however a check of seismic records indicated no earthquakes had occurred during the previous 48 hours. Thus, seismic activity was ruled out as a contributing factor.

Cause of the Rockfall--The major contributing factor was the adverse structural geology involving the wedge geometry combined with the overhanging cross section (figure 10-3). It is likely that seepage pressures, water pressures in joints and ice-jacking caused by freeze-thaw cycles also contributed to the failure. A portion of the overhang did not fail.

Rockfall Mitigation--It was considered that an extension of the rockfall of the overhang area could occur. The slope should be scaled of all loose rock and potential wedge or planar failures, the surface water above the crest should be diverted, the trees near the crest should be removed and the inner ditch should be deepened about 3 feet (.9 meters) or a concrete Jersey barrier should be placed along the shoulder

Figure 10-3 - The failure comprised a wedge from the overhang area. Some of the overhang and a further wedge remains. This rock should be removed. Note the flow of water over the lower slope. This water should be diverted behind the crest to reduce seepage, water pressures, and freeze-thaw ice-jacking. Trees should be removed along the crest *(Courtesy Alaska Department of Transportation)*.

to increase catchment volume. All of the rock slopes in Keystone Canyon should be inspected for potential future rockfall.

The litigation was settled out of court for a fraction of the original claim.

10.3. KEYSTONE CANYON, ALASKA

An old tunnel constructed on a curve near mile 14 north of Valdez was having roof stability problems and posed a traffic hazard for large, long trucks. The Alaska Department of Transportation decided to realign the highway to bypass the tunnel. This resulted in two bridges crossing the Keystone River with a 300 foot (91.5 meter) high sliver rock cut in between.

The rock was bedded and dipped uniformly at about 65° toward the river (figure 10-4). Numerous exposed surfaces were light colored, indicating blocks and slabs had recently fallen from the face.

Two design options were considered.

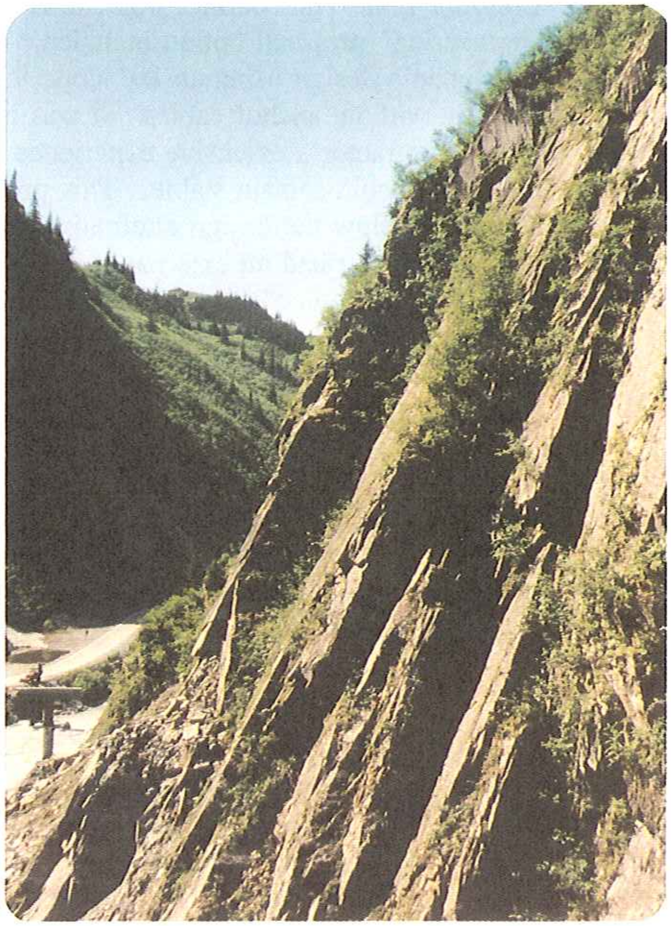


Figure 10-4. A 300 foot (91.5 meter) high rock cut was proposed to develop new alignment. Note the bridge pier in the river. The bedding in the rock dipped at about 65° toward the river. The design recommended a wide catch ditch at the toe with the upper slope to be excavated to the dip angle with no benches. Scaling and bolting and drain holes were recommended as excavation developed.

- Design the slope at the dip angle with a wide catchment ditch at the bottom. Use controlled blasting to minimize slope damage. Allow for rock bolting of some slabs and incorporate drainage holes at every bench elevation to minimize pore water pressures.

- Develop the slope at 80° and install tensioned grouted anchor cables every 20 feet (6.1 meters) to stabilize the undercut joints in the slope. This greatly reduces rock excavation quantities.

Estimated costs of the two options were comparable. However, the long-term stability of option 1 was considered more favorable and was recommended.

Contract Call--The contract was called with a "value engineering" proposal option included. One contractor developed a design using an 80° slope that undercut the bedding without anchor cables. It was claimed that based on the contractor's extensive experience in rock excavation, the slope would remain stable. This proposal bid was about $500,000 below the design alternative and other bidders. The department had an external consultant review the "value added" design, which it subsequently accepted. The original designer was not advised of the change. The project geologist expressed his concern to senior highway staff, but to no avail.

Construction--The contractor commenced rock drilling and blasted the upper slice using controlled blasting (figure 10-5). During the initial cleanup with a Caterpillar tractor and hand scaling, a major rockfall occurred that killed the Cat operator and his father who was scaling (figure 10-6).

The writer received an urgent telephone call to travel to the site and to recommend future work. The inspection revealed failures had occurred along the adverse dipping structure.

A recommendation was made to construct the cut as per the original report and design--to excavate down the dip slope of the bedding. This was accepted. The original contract was cancelled and the project rebid and called.

Reconstruction--Due to difficult access, the pioneer cut had to be developed using a helicopter to supply men, equipment, and materials. The final slope drilling was performed with closely spaced holes following the dip angle. Twenty-foot (6 meters) bench depths were used to ensure better control of drilling, blasting, scaling, and occasional rock bolting. Each new slope face had to be scaled and stabilized before drilling could commence below. A catch ditch wide enough to allow equipment access was designed at the slope toe. The slope excavation and stabilization was completed without incident. Figure 10-7 shows the rock slope following completion.

The family of the deceased workers instigated a law suit against the State. After three separate trials, they were awarded a settlement of about $2 million.

Figure 10-5. The initial upper cut developed at about 80°. This undercut the joints and led to instability *(Courtesy Alaska Department of Transportation).*

Figure 10-6. Rockfall occurring during cleanup after the first blast. Note the Caterpillar cab partially buried. The operator and his father who was hand scaling were killed. The family instituted a claim and were ultimately granted a substantial award *(Courtesy Alaska Department of Transportation).*

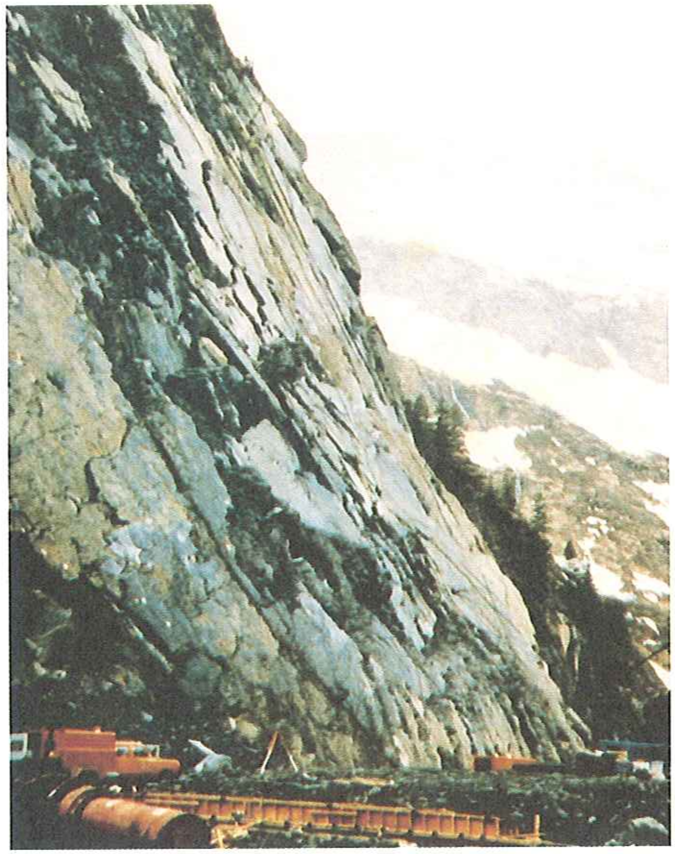


Figure 10-7. Rock slope after completion of excavation to the dip slope. Controlled blasting was used, loose rock was scaled, some rock bolts were installed, and drain holes were drilled into the rock. A wide catch ditch was developed.

Figure 10-8. Station wagon struck by large rock (lower right) on the freeway. The driver lost control and drove across the wide ditch up to the rock face. Two childern were killed and six others were injured *(Courtesy Oregon Department of Justice)*.

10.4. COLUMBIA RIVER GORGE, OREGON

In October, 1984, a rock fell from a cliff about 200 feet (61 meters) above the four-lane freeway I-84 near mile 53 east of Portland and landed on a station wagon. Two children were killed (figure 10-8). The family initiated a law suit against the State of Oregon. The writer was retained to evaluate the cause and engineering responsibility.

Site Conditions--The freeway was constructed in 1965-66. Figure 10-9 shows an oblique air photo of the rock slope and the origin of the rockfall. The eastern end of the rock cut was excavated with one catch bench partway up the slope. The west third of the rock slope was only excavated at the lower portion. A natural unexcavated bench existed. The upper 150 feet (45.8 meters) of the west third of the cliff was not encroached upon by original construction.

The upper 100 feet (30.5 meters) of the slope averages about 60° but is very irregular. About mid-height, a flatter natural talus slope at about 35° existed followed by a steeper section with a natural bench partially filled with talus rock. The lower excavated slope was developed at about ¼:1.

The ditch section was unusually wide (47 feet or 14 meters) from the toe of the shoulder to the toe of the slope.

The rock comprises basalt flows, which are weathered to moderate depth where they have not been excavated.

Heavy rainfall of 1.5 inches (38mm) occurred during the 24-hour period prior to the fall.

Rock Fall--Estimates of the rockfall ranged up to about 120 yd^3 ($92m^3$). The origin of the rockfall from the cliff is shown in figure 10-10. Some of the rock bounced to a side gully, some rock was caught in the two talus portions of the slope. The majority fell into the ditch. Some rock blocks bounced more than 60 feet (18 meters) onto the traffic lane with one hitting the car. In order to do this, the rock must have had substantial momentum and bounced from a very hard rock exposure. The distance of the bounce was unusual.

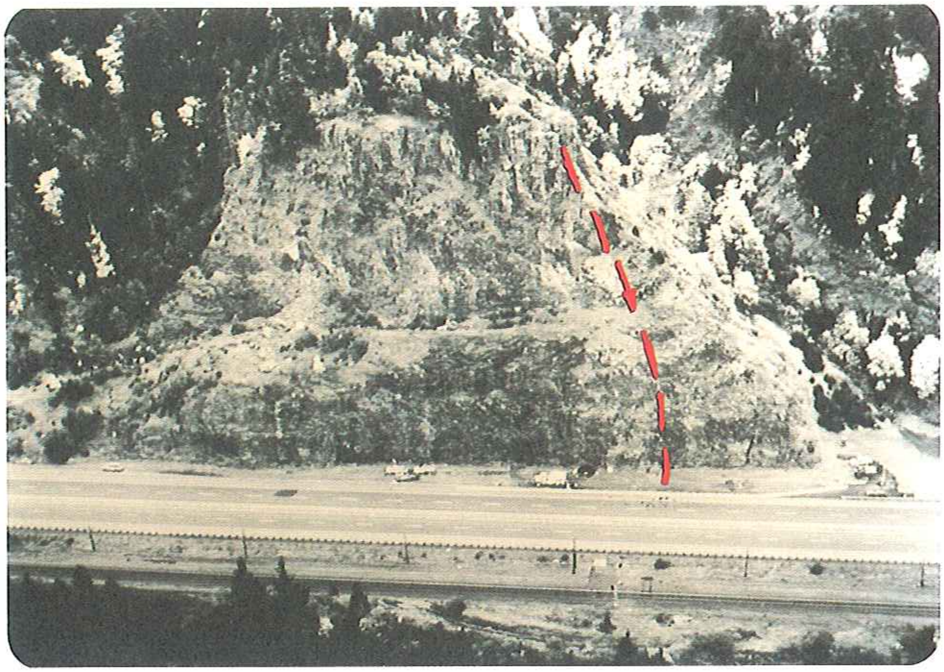


Figure 10-9. High rock cliff on I-84 east of Portland, Oregon. A large rockfall fell from the upper cliff, rolled down the slope, and several large rocks bounced out onto the freeway across a 47-foot (14 meters) wide ditch. One block struck a station wagon. Note the constructed bench along the easterly two-thirds of the cut. The west end of the cut is generally natural slope. The black spots on the highway are patched rockfall indentations. Note the railway north of the freeway *(Courtesy Oregon Department of Justice)*.

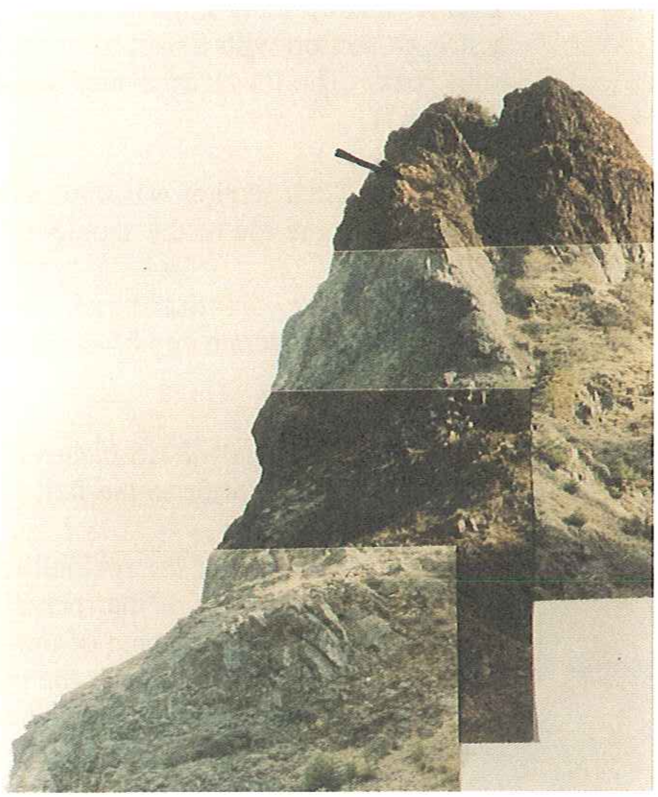


Figure 10-10. Composite photograph of cliff looking east. The rockfall originated from the fresh face near the top of the cliff. Some of the rock bounced more than 60 feet (18 meters) horizontally onto the freeway (*Courtesy Oregon Department of Justice*).

At this time the State of Oregon had a rock rating and prioritization program in place. This area was rated No. 11 out of 21 priority locations in the region. Depending on funds for stabilization it would have been due for work in several more years.

Cause of the Rockfall--The rock fell from a high portion of the cliff well away from the original construction. The location is more than 300 feet (91.5 meters) from the highway, 400 feet (122 meters) from the railway line, and no earthquake had preceded the fall, thus vibration was not considered a cause.

The heavy 1.5 (38mm) inch rainfall that preceded the fall was believed to have been the triggering factor. Water pressure buildup in tension cracks or joints could have precipitated the failure. Natural ongoing weathering would be a contributing factor.

Engineering Interpretation--It was concluded that the Oregon Department of Transportation met the Standard of the Industry for freeway and ditch design at the time the freeway was constructed. It was further advised that the State of Oregon's prioritization program was evidence of a responsible concern for long-term stability.

It was concluded that the rockfall was due to natural causes, namely rainfall-induced water pressures and natural weathering. The litigation was settled out of court for a nominal amount.

A stabilization program carried out after the accident included laying back the slope above the bench and placement of gabion and concrete binwall rockfall catch barriers along the bench crest and highway shoulder respectively.

10.5. NEW YORK STATE THRUWAY AUTHORITY

In 1980 a major rockfall accident that killed the passenger occurred on the New York Thruway. The accident resulted in a rock slope survey that identified 35 sites for mitigation. Temporary procedures including fences, scaling, and draped mesh were used at many of the locations until permanent stabilization could be put in place. At four sites, a tieback wall up to 36 feet (8 meters) high was constructed to create a catchment area at the top of the wall. This procedure was required to limit disruption of traffic along a section of highway with an ADT of more than 50,000 vehicles and speeds in excess of 70 mi/h. Conditions for this project describe Rockland County, 30 to 40 miles (48 to 64km) north of New York City.

Site Conditions--The highway runs along the eastern flank rock ridges of the Ramapo River Valley. The ridges comprise highly contorted gneissic rock with major N-S trending folds. Weathering is extensive. The project comprises 6 lanes with a grade separation of between 5 and 15 feet (1.5 to 4.6 meters) between roadways. The highway was constructed between 1953 and 1955 before controlled blasting was used. Slope angles ranged from 1H:3V to local overhanging rock. All slopes showed signs of considerable over-blast damage frequently extending 25 to 30 feet (7.6 to 9 meters) into the slope. Slope heights extended up to 100 feet (30.5 meters). An interstate park existed beyond the right-of-way.

Investigations--Estimates were made to evaluate scaling and recutting. Constructibility presented severe problems. The disposal of excavated material, long permit time to obtain right-of-way and access through the park, the need for maintenance and protection of traffic during stabilization, led to the evaluation of construction of a wall high enough to create adequate catchment.

Detailed geologic and structural studies were performed. These concluded that there was adequate foundation support for a wall. Field and photogrammetric cross sections were obtained. Past maintenance and rockfall information was reviewed. Maximum rockfall size expected was approximately 10 yd^3 ($7.7m^3$).

The evaluation indicated that a wall scheme was feasible and four design criteria were established.

- The structure must have a narrow foot print and still retain a full width shoulder. The slope setback varied between 15 and 20 feet (4.6 to 6 meters) from the traffic lanes.

- The structure must provide a high degree of security against slide debris reaching the pavement. High energy rockfalls were expected.

- The structure must combine a high level of structural stability with an aesthetically pleasing appearance and low maintenance. The structure must resist salt and other de-icing chemicals.

- The system must be able to be constructed from a limited staging area. The maximum lane shift available was 10 feet (3 meters). This resulted in a maximum work zone width of 23 feet behind the concrete barrier.

Construction--A wall system was selected using 12 x 12 x 84 steel tied back soldier piles set on 12-foot (3.7 meters) centers into 24-inch diameter (610mm) predrilled holes at least 6 feet (1.8 meters) into rock (figure 10-11). The piles were spliced for ease of installation of both the piles and tiebacks. Tiebacks were 1 3/8-inch (36.8mm) grade 150 Dywidag epoxy coated bars. Vertical spacing of the tiebacks was 8 to 12 feet (2.4 to 3.7 meters). Underdrain pipes were installed at the base of the wall.

Wall modules were 4 by 24 foot (1.2 by 7.32 meter) reinforced precast concrete units with each unit cast with two 21-inch round pile holes (figures 10-12 and 10-13). Thickness varied from 17 to 32 inches (432 to 813mm). All units have an exposed aggregate architectural finish. (figure 10-14). Following placement of units the annular space around the pile was filled with concrete to provide load transfer to the piles and corrosion protection.

Backfill to within 5 feet (1.5 meters) of the top of the wall was 1 1/2-inch (38mm) crushed stone. A geotextile membrane was then placed on top, covered with select granular material and an interceptor drainage ditch was constructed. This was covered with select granular material.

A Brugg cable net catchment fence 12 to 18 feet (3.7 to 5.5 meters) high was designed for installation at the top of the walls (figure 10-15). The fence panels have an 8-inch (203mm) square mesh size woven with 1/4-inch (6mm) wire rope and 5/16-inch (7mm) border rope. The panels are hung on 8 x 8 x 48 wide flange posts on 20-foot (6 meter) centers. The posts are tied back at both the top and post anchors. They were designed to act independently of the wall. The fence is set back 4 feet (1.2 meters) from the face of the wall to allow the fence to deflect.

Figure 10-11. Steel piles installed in predrilled and cased foundation holes. The Jersey barrier confines traffic to the right to provide work space *(Courtesy New York Thruway Authority)*.

Figure 10-12. Precast concrete panels were slid over the steel piles and tied back with Dywidag bars grouted into the rock behind *(Courtesy New York Thruway Authority)*.

Figure 10-13. Precast concrete panels being placed by sliding down over the steel piles. Note the exposed aggregate facing. This construction was timed during low traffic periods *(Courtesy New York Thruway Authority)*.

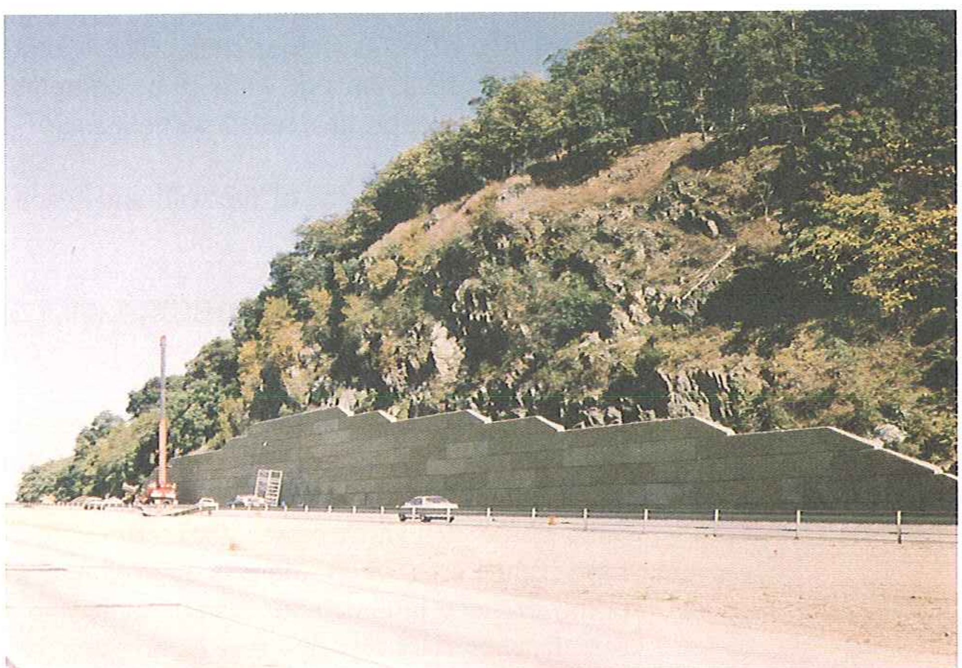


Figure 10-14. Completed concrete wall. Note the pleasing lines and appearance *(Courtesy New York Thruway Authority)*.

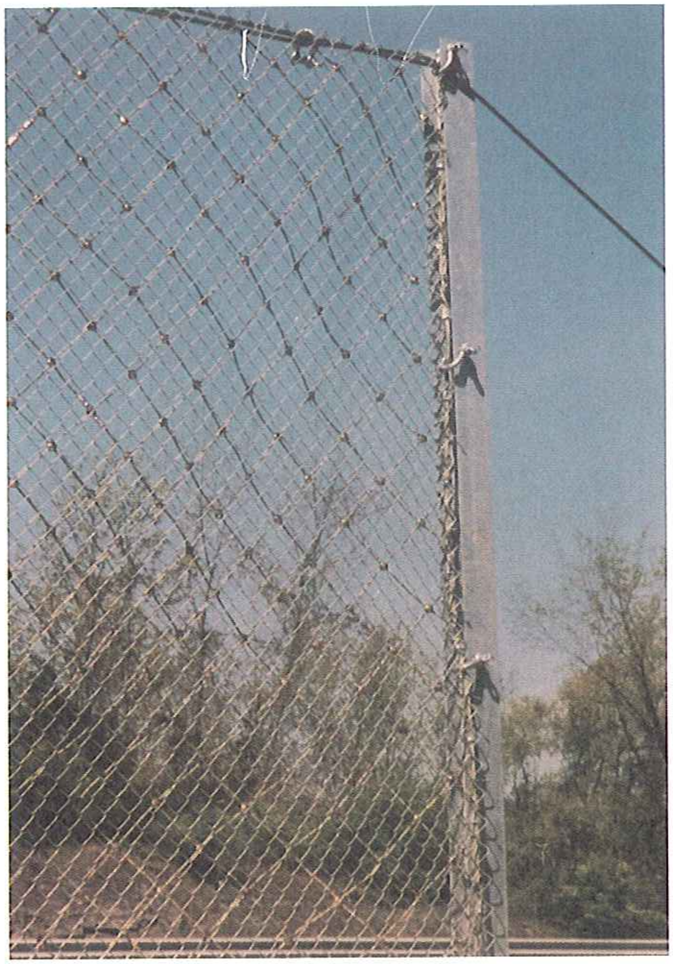


Figure 10-15. Wire mesh catch fence showing steel column, mesh, clasps, and connections. This fence was developed as a construction fence *(Courtesy New York Thruway Authority)*.

A contract was let in 1989 for \$13.8 million for the construction of walls at four sites. The contract called for 110,000 ft^2 (10,230m^2) of walls to be completed by May 1990. Construction proceeded without major problems.

The performance of the wall and fence design will be evaluated periodically.

10.6 EAGLE FALLS PROJECT, LAKE TAHOE, CALIFORNIA

Site Conditions--The following summarizes the California Department of Transportation's Eagle Falls Rockfall Project in El Dorado County. The project site is located along State Route 89, which passes above Emerald Bay in the Lake Tahoe area where there is approximately 0.6 miles (1km) of steep mountainous terrain subject to rockfall. Maintenance staff reported that there were frequent rockfalls that caused automobile accidents and hazards to maintenance forces.

In response to maintenance staff requests, California Department of Transportation engineering geologists performed geologic and rockfall studies at the site. Included in the study was a detailed rockfall investigation that consisted of field mapping to identify rockfall locations and characterization of rockfall sizes and frequency. Environmental investigations based upon the visual impact assessment procedures developed by the FHWA were performed following the geologic/rockfall investigation.

After careful analysis, several rockfall mitigation measures were selected that satisfied engineering geology and environmental concerns. Because of the steep terrain and proximity to State and Federal Resource lands, all mitigation designs had to be constructed within the existing narrow right-of-way. The excavated slopes were in glacial till with numerous large boulders (figure 10-16). Typical rockfall sizes ranged between 2 to 10 feet (.6 to 3 meters) in diameter and were falling from as high as 100 feet (30.5 meters). Because of the size of the rocks, potentially high impact energies, and limited space, many rockfall measures were not possible. Only stabilizing problem areas and protecting the roadway were considered feasible.

Design--Stabilizing some problem areas was accomplished by covering the slope with rock armour (figure 10-17). This was carried out on slopes as steep as 1½:1. On steeper slopes, tiered tie-back walls were constructed and slope armour placed around the walls. At several locations the slopes were covered with jute mesh and seed.

Protection measures were accomplished by installing rockfall catchment walls to increase rockfall catchment widths and depths (figures 10-18 and 10-19). On the uphill side, the walls were covered with timbers and the ditch was partially filled with energy-absorbing fill. In most locations, viaducts were constructed to move the roadway out far enough to get adequate catchment width (figure 10-20).

Prior to construction, scaling and trimming removed loose rocks and large rocks where possible.

Five different rockfall mitigation designs were used. Difficult access and moderate to high rockfall energies characterize these locations.

Figure 10-16. Bouldery glacial till slope near Lake Tahoe, California. During heavy rains or snow melt, the large boulders can be undermined by erosion and ravel down the slope. The worker is readying a trim blast to break the rock to small sizes, which will be scaled *(Courtesy Caltrans)*.

Figure 10-17. Boulder till slope covered with rock armour to provide stability Slope angle 1½:1 *(Courtesy Caltrans)*.

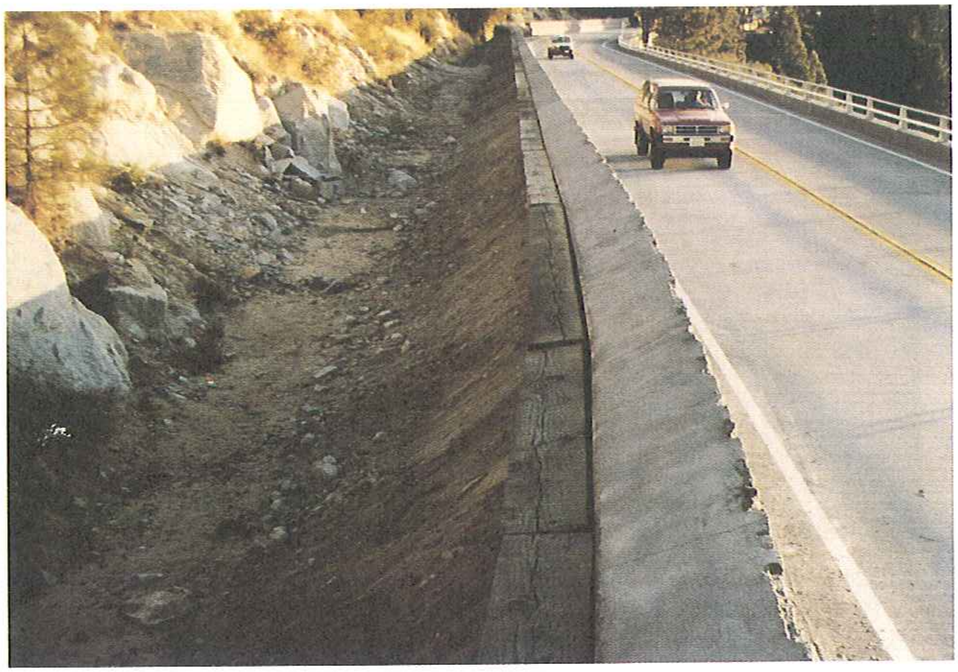


Figure 10-18. Catch ditch below boulder till slope. A concrete wall is backed with soil and timbers as a buffer *(Courtesy FWHA/Caltrans)*.

Figure 10-19. Architecturally faced catch wall below a boulder till slope. This is the front face of figure 10-18 *(Courtesy FWHA/Caltrans)*.

Figure 10-20. The highway moved out to provide a wider inner catch ditch. This required a concrete viaduct structure.

Environmental Evaluation--The environmental evaluation required approval from various environmental agencies because of the scenic location. Considerable input was received from county, State, and Federal environmental personnel, as well as from the public. Through public scoping meetings, Caltrans engineering geologists, engineers, landscape architects, and environmental planners were able to develop a project that minimized impacts to the area. This included landscaping and coloring the concrete, as well as using patterned architectural panels. The tiered tie-back wall/rock armour combination and the viaduct/rockfall catchment wall combination dramatically reduced wall heights and thereby effectively reduced the visual impact on this scenic area. Also the rock slope armour was hand placed around existing vegetation resulting in a natural looking slope, particularly when local rock was used.

Planning began in 1985. Construction began in September 1989, and was completed in April 1991. The rockfall mitigation measures used in this project have very positive design features that allow their use and approval in sensitive, steep, mountainous terrain.

The detailed rockfall investigation and the participation of engineering geologists throughout this project proved to be of considerable value and is recommended on rockfall projects.

10.7. GAVIOTA PASS ROCKFALL PROJECT, CALIFORNIA

This project involves investigations and construction plans for the California Department of Transportation's Gaviota Pass Rockfall Project in Santa Barbara County. A portion of California State Route 101 is located within the pass where there is approximately one mile (1.6km) of steep mountainous terrain that is subject to rockfall. Numerous auto accidents caused by rockfall had been recorded in the area. At the request of maintenance staff the California Department of Transportation Engineering Geologists performed rockfall studies which resulted in the project. Included in the studies were detailed geologic rockfall investigations.

Investigations--The investigations consisted of detailed field mapping to identify rockfall locations and to characterize rockfall sizes, frequency, and site accessibility for construction. The investigation also included rock rolling tests at selected locations to determine rockfall velocities, trajectories, and kinetic energies. Rock rolling field data were also used for computer modelling of rockfall behavior at every potential rockfall location. Computer analysis enabled engineering geologists to model hundreds of rockfalls. The computer model used was the Colorado Rockfall Simulation Program (CRSP). Environmental investigations, based upon the visual impact assessment procedures developed by the FHWA, were performed following the geologic/rockfall investigations.

Nine sites were identified as having rockfall problems. Rockfall energies ranged from 15 foot-tons to 70 foot-tons. Typical rockfall sizes are between 1 to 3 feet (.3 to 2.7 meters) in diameter. After careful analysis, several rockfall mitigation measures were selected that satisfied engineering geology and environmental concerns.

Design--The designs were protection and control measures and included flexible rockfall barriers, draped wire mesh, and anchored wire mesh (figures 10-21 to 10-24). Scaling was performed prior to all work. Because of the steep and

Figure 10-21. Typical topography in Gaviota Pass near Santa Barbara, California. This slope was covered with rock net at the intersection of the cut slope and natural slope *(Courtesy Caltrans)*.

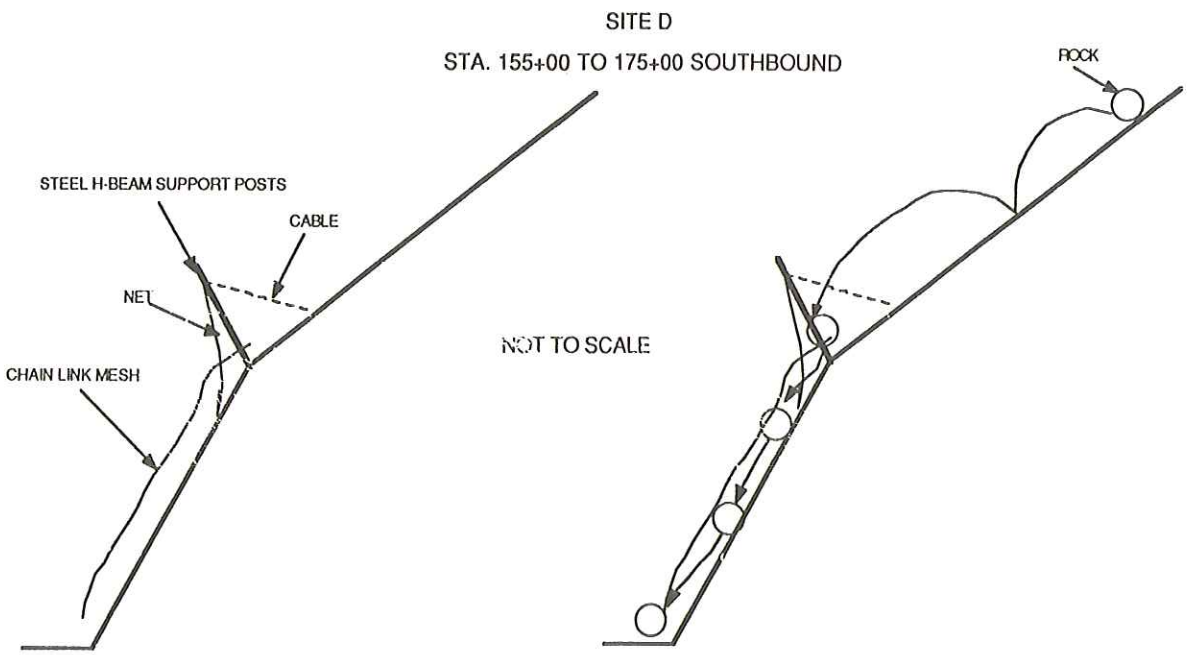


Figure 10-22. Rock net at the intersection of the cut slope and the natural slope. The bottom of the net is attached or overlapped with a chain link mesh drain over the cut slope *(Courtesy Caltrans)*.

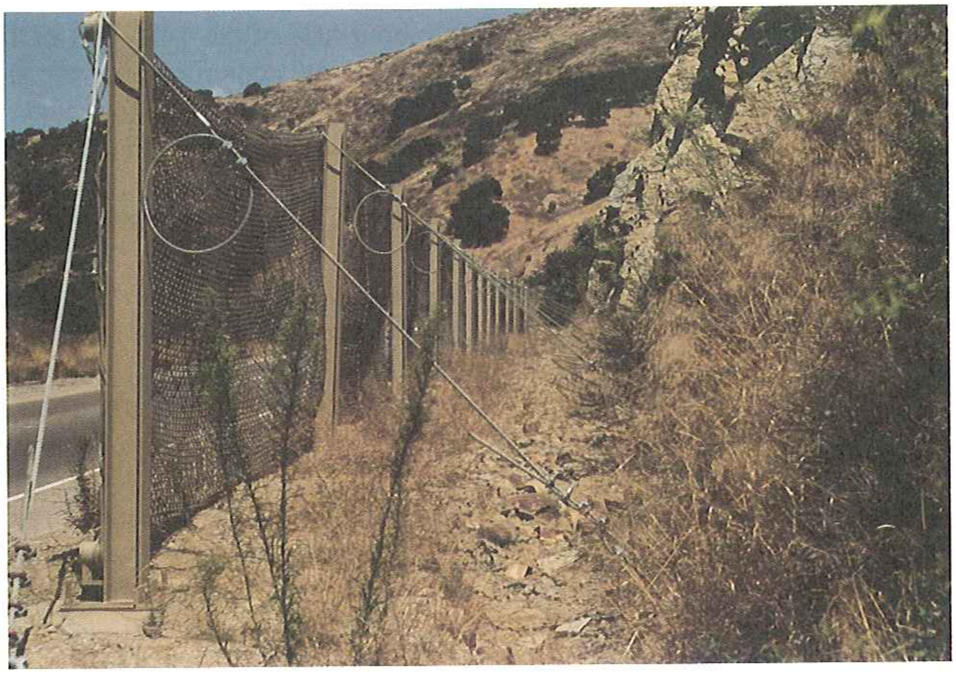


Figure 10-23. Brugg woven wire rope rock net. Posts and chain link mesh are earth tone colors. *(Courtesy Caltrans).*

Figure 10-24. Multi-level woven wire rope nets. Posts and netting are earth tone colors. *(Courtesy Caltrans)*

narrow terrain, about half of the 1400 feet of flexible barrier installed had to be constructed on the slope. The other 700 feet (213.5 meters) was constructed near grade on benches 3 feet above the roadway. The flexible barriers used were woven wire rope rock nets provided by Brugg Cable Products and L'Entreprise Industrielle.

Rock net versatility is emphasized with installations above cut slopes on near-vertical to 1:1 natural slopes and on roadside benches. Five different types of rock net installations were constructed. Difficult access and moderate rockfall energies characterize these locations.

Controlling measures utilized were draping hexagonal wire mesh over the cut slopes and in one area anchoring the mesh to the slope face. In two installations the drapery was attached to the bottom of the upslope nets. This design uses the nets to catch falling rocks, which then can move downslope behind the drapery to road level where the rock can be safely removed.

The environmental evaluation required approval from various environmental agencies because of the scenic location. County, State, and Federal environmental personnel, as well as the public, provided considerable input. Through the use of public scoping meetings, Caltrans engineering geologists, engineers, landscape architects, and environmental planners were able to develop a project that minimized impacts to the area. This included landscaping and coloring the rock net posts, chain link, and drapery with earth tones. Color coordinating the mesh was crucial in receiving environmental acceptance.

Planning began in 1986. Construction began in October 1992, and was scheduled for completion in April 1993. The rockfall mitigation measures used in this project have very positive design features that allow their use and approval in sensitive, steep, mountainous terrain.

The detailed rockfall investigation and the participation of geologists throughout this project proved to be of considerable value and are recommended on rockfall projects.

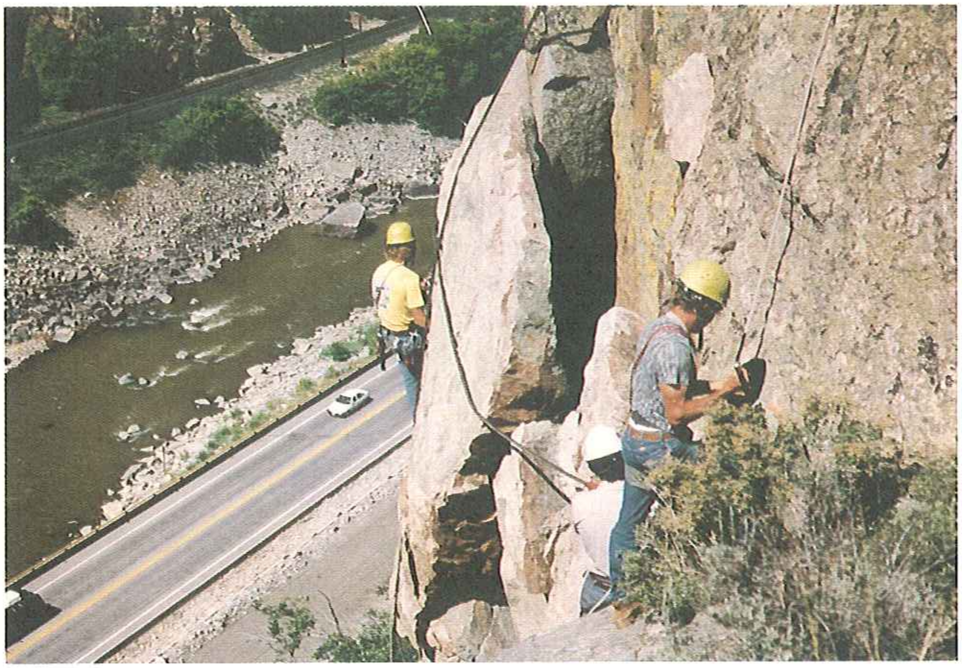


Figure 10-25. Large block being cabled about 150 feet above the freeway. There was fear the drilling for rock bolts would cause failure due to vibration and that corrosion of bolts would occur *(Courtesy Colorado Department of Transportation).*

10.8. GLENWOOD CANYON, COLORADO

Site Conditions--A large rock about 20 feet (6 meters) high, 15 feet (4.6 meters) wide and 3 to 4 feet (.9 to 1 meters) thick had separated from the main rock about 150 feet above the highway (figure 10-25). The base was fracturedand a portion of the support rock was missing. The separation was believed to be due to annual ice-jacking. The movement indicated a toppling-type failure would ultimately occur. The inner highway ditch was too narrow to catch the rock if it fell. Rockfall analysis using the rockfall simulation program indicated if the rock fell it would reach the highway.

Stabilization--Rock bolting was not considered appropriate because of the separation and the fractured nature of the rock.

Any rock bolts would be exposed to the elements in the voids. In addition, there was concern that the vibration of drilling could cause failure.

The large size of the rock ruled out scaling. Even if the block broke into several large blocks, there was a threat to highway structures below. The cost of controlled drilling and blasting was estimated to be very high, particularly since protection measures and traffic control would be required. Widening the ditch was not considered feasible because of the volume of rock that would require removal.

Design--A cable lash scheme was reviewed and considered appropriate. The loads calculated to prevent toppling were modest. Because of the fractured nature of the rock mass, the design incorporated three equally spaced horizontal cables with a vertical cable from top to bottom. Anchor points were drilled and #10 epoxy-coated Grade 60 Dywidag bars were grouted with Celtite epoxy resin. The cables were specified to be 7/8-inch (21.6mm) wire rope with factory installed thimbles. The nominal breaking strength was 68 kips with a safety factor of 4. Galvanized, 33½ closed length turn buckles rated at 15 kips at a safety factor of 5 were specified to remove slack in the cables after installation. Large galvanized shackles were used at the anchor points behind the epoxy coated plates, washers and nuts to attach to the turn buckles.

Construction--The site could not be reached by cranes so the work had to be done by hand. Technical rock climbing was required. The anchor holes were marked and exact cable lengths were measured prior to construction. The anchor holes were drilled by a two-man crew using a hand sinker and a 185 CFM compressor. The turn buckles were turned by hand and cheater bar until all slack was removed (figures 10-26 and 10-27). The cables were retightened a month after installation. The final cable-lashed rocks are shown in figure 10-28. Total cost for the cable lash stabilization was $9,000. Seventy-six percent of the cost was labor costs.

10.9. GLENWOOD CANYON, COLORADO

Site Condition--A large slab of quartz diorite (25 by 20 by 8 feet or 7 by 6 by 2 meters) weighing about 200 tons (181mg) rested on slope of 45° to 50° (figure 10-29). The geometry was controlled by joints. A one-foot thick weathered zone existed along the lower portion of the rock slope. This zone is weak and partially eroded. The rock was cantilevered in place at 45° with a one-foot gap extending upwards 15 feet (4.6 meters) separating the slab from the unweathered lower rock mass. The weathered zone showed signs of crushing. A horizontal joint existed at the top of the slab where it contacted the rest of the rock

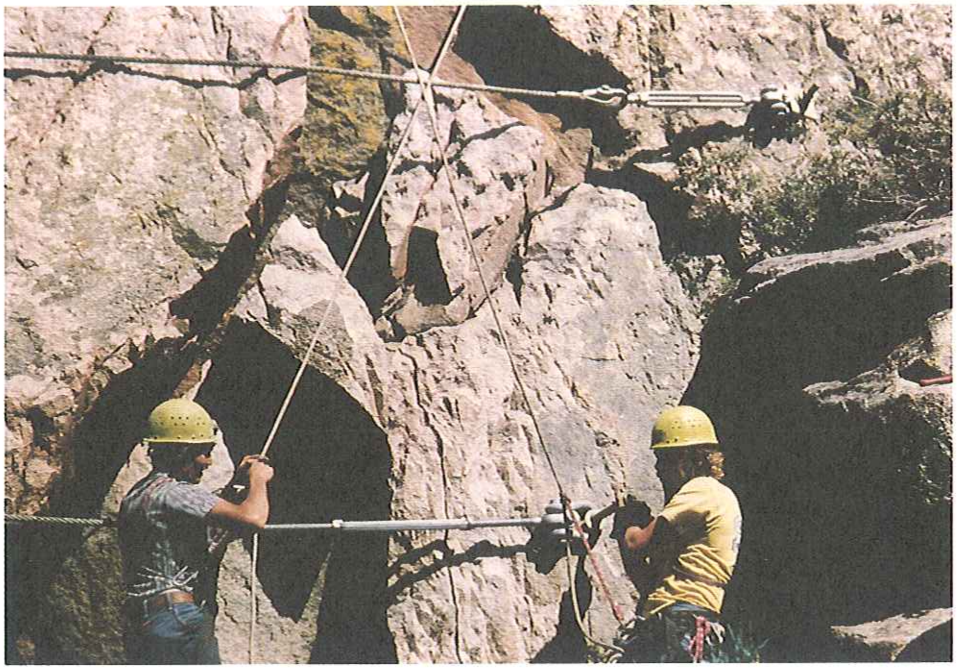


Figure 10-26. Installation of anchors and lash cables by technical rock workers *(Courtesy Colorado Department of Transportation).*

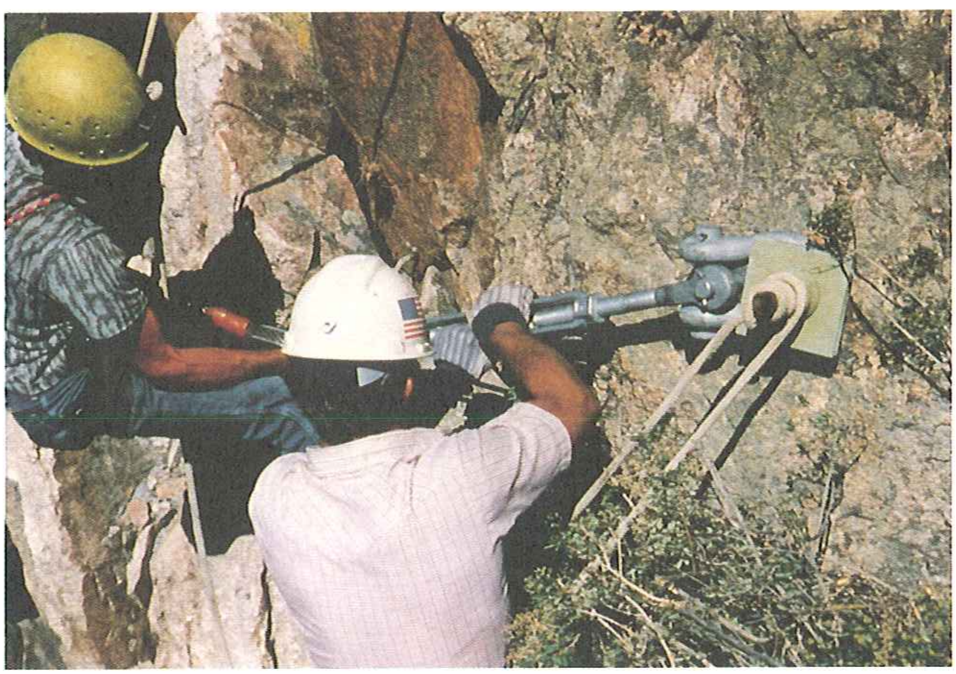


Figure 10-27. Tightening of turnbuckles. These were tightened one month after installation *(Courtesy Colorado Department of Transportation).*

Figure 10-28. Completed lashed rock with vertical and horizontal cables *(Courtesy Colorado Department of Transportation).*

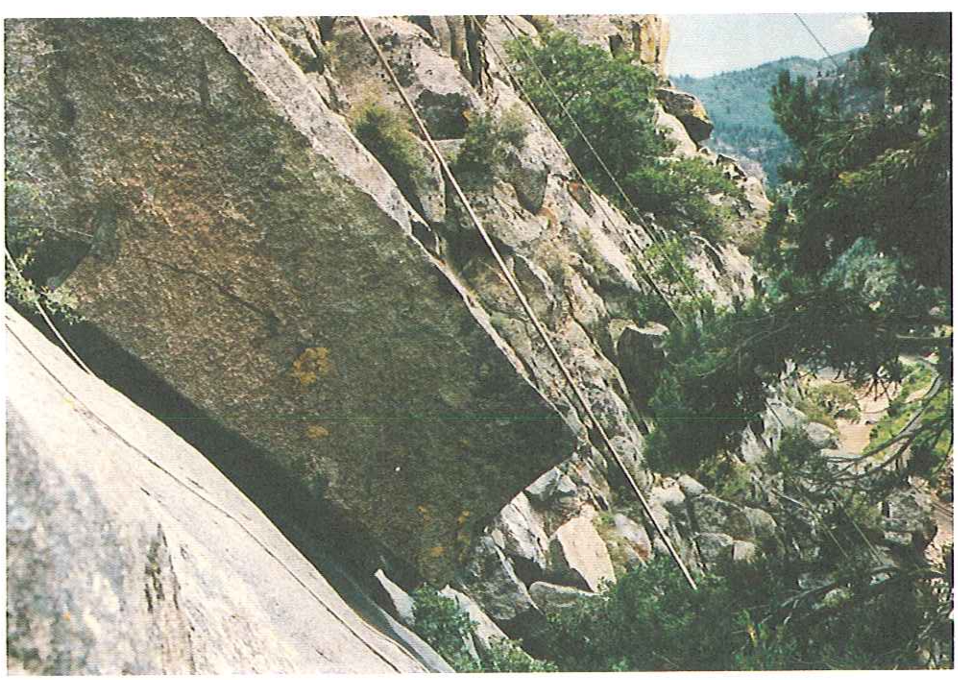


Figure 10-29. Unstable slab resting on a 45° dip slope. The site is about 300 feet (91.5 meters) above the Glenwood Canyon highway *(Courtesy Colorado Department of Transportation).*

mass. There was serious concern that the horizontal joint would separate and the slab would slide on the weathered zone and fail.

About 150 feet (45.6 meters) to the east of the unstable area, a series of irregular rock blocks had become detached and offset to the point of being potentially unstable. Frost wedging had caused the movement. These blocks were smaller but supported a large 9 by 12 by 7 foot (2.7 by 3.7 by 2 meters) block located about 300 feet (91.5 meters) above the freeway. The lower smallest block had displaced about 2.5 feet (.8 meters). If it were to fail the rest would follow. This key block area required support.

Design--The sites were about 300 feet (91.5 meters) above the highway. Removal of the unstable rocks by blasting was possible but dangerous because of potential instability from the vibration and the threat of damage to existing highway structures below. The cost to protect them, plus the required traffic control and high cost of drilling, blasting, scaling, and cleanup resulted in alternative consideration. Passive stabilization procedures were considered. Cable lashing and wire mesh schemes were not considered appropriate due to the high loads involved.

The design chosen was concrete buttresses anchored with epoxy coated No. 10, 150 Grade Dywidag bars into competent rock. The anchor bars were then incorporated into a rebar cage fabrication (figure 10-30). Forms were constructed that would retain concrete against the unstable rock.

Construction--Class D 4500 psi concrete with 6-inch (152mm) slump was poured and vibrated to achieve suitable compaction and density. At no time was the unstable rock mass disturbed. Crane access was impossible due to the height. Technical rock climbing skills were required to access the sites and all site work was performed by hand. Holes were drilled by a two-man crew using hand sinkers and a 185 CFM compressor. Bars were grouted using Celtite epoxy resin cartridges. A generator was brought up to the site by cable tram to power carpenter tools and the concrete vibrator. Fourteen yd^3 of concrete was delivered by helicopter in ¼ yd^3 buckets (figure 10-31).

One completed anchored buttress is shown in figure 10-32.

The total costs for the buttresses was $21,000, of which 80 percent was labor costs.

10.10. GLENWOOD CANYON, COLORADO

Site Conditions--An unstable rock mass was identified about 300 feet upslope near Station 267 on the Glenwood Canyon highway. An overhanging block of quartz diorite was perched on a 50° inclined joint surface. The underlying slab was also inclined on another joint surface of the same set (figure 10-33). Behind the rock was a regolith soil mantle with vegetation and a lone pine tree. An examination of the exposed joint contact below revealed some staining caused by surface water flow. The geometry of the rock was measured. An analysis of stability using a cohesion = zero and angle of friction 1 indicated a safety factor of 0.7. It was apparent that the rock was being held in place by the surface roughness along the joints.

Design--Removal by blasting or the installation of rock bolts were considered to be the only reasonable mitigation options. Blasting was rejected because of potential damage to structures below, the need for traffic control, and the estimated high cost. Also, a prominent ugly scar of blast debris would be left on the rock slope.

A rock bolt design was developed to support the top block and the underlying slab. Fifteen epoxy coated No. 10, 150 Grade Dywidag bars were installed, anchored, and tensioned to 90 Kips to obtain a satisfactory safety factor. Bar length was specified to be 20 feet (6 meters) minimum to develop an adequate length below the potential failure plane.

Construction--Crane access was not possible so all drilling and installation had to be accomplished by hand by technical rock climbers. A cable tram was established to facilitate the transportation of materials to the work site. Holes were drilled by a two-man crew with hand sinkers using a 185 CFM compressor (figure 10-34). The holes were begun with a diameter of 2 inches (51mm) reducing to 1 7/8 inch (45mm) and 1 3/4 inch (44mm). Drilling rates reduced substantially with depth.

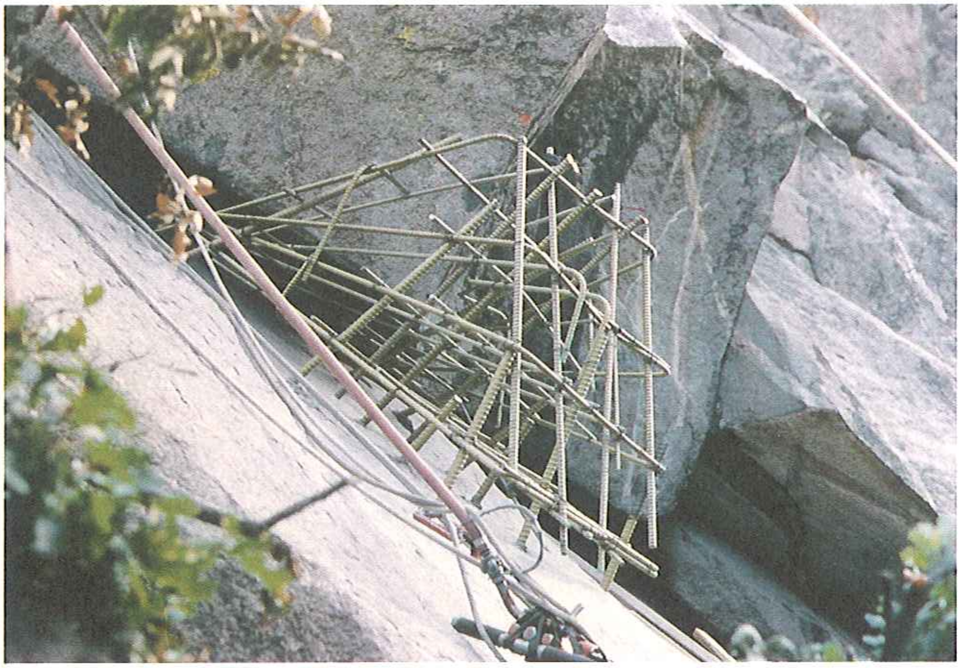


Figure 10-30. Fabricated rebar cage tied to the grouted anchor bars. Note the open joint below the large block *(Courtesy Colorado Department of Transportation)*.

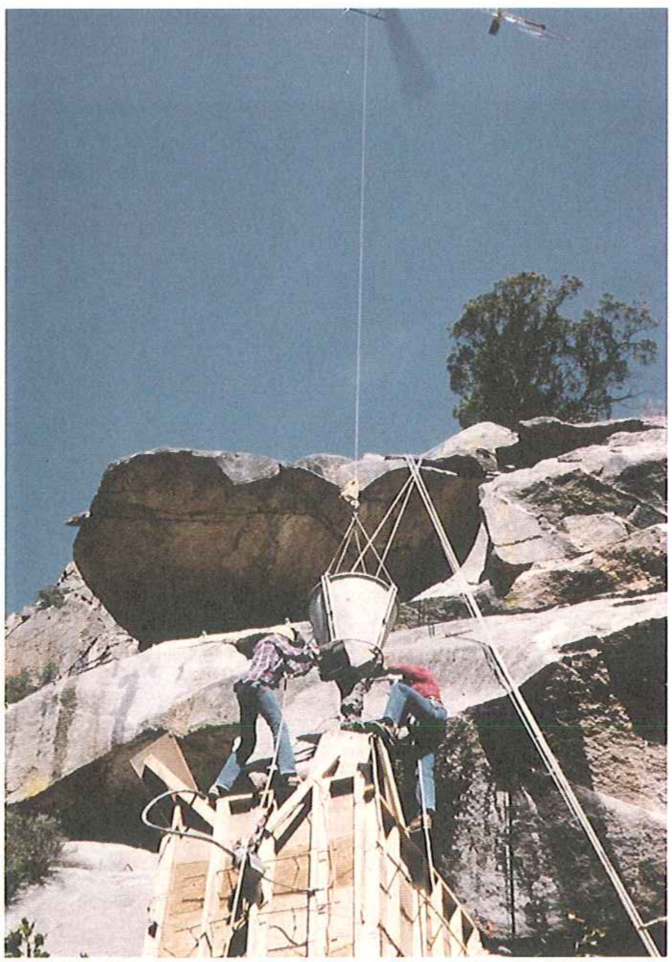


Figure 10-31. Helicopter lifting the bucket above the form to pour concrete. The bucket holds ¼ yd^3 of high slump class D concrete. The concrete was vibrated to obtain the required density *(Courtesy Colorado Department of Transportation)*.

Figure 10-32. Completed anchored concrete buttress *(Courtesy Colorado Department of Transportation)*.

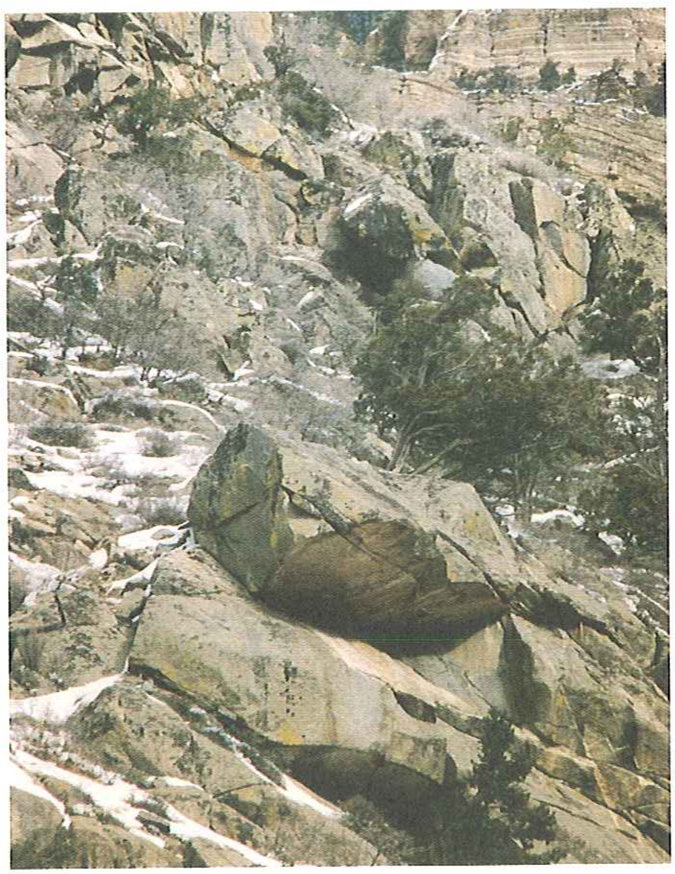


Figure 10-33. Large rock blocks that are potentially unstable. The 15 to 20 foot (4.6 to 6 meter) long corrosion protected tensioned bars were installed to provide stability *(Courtesy Colorado Department of Transportation)*

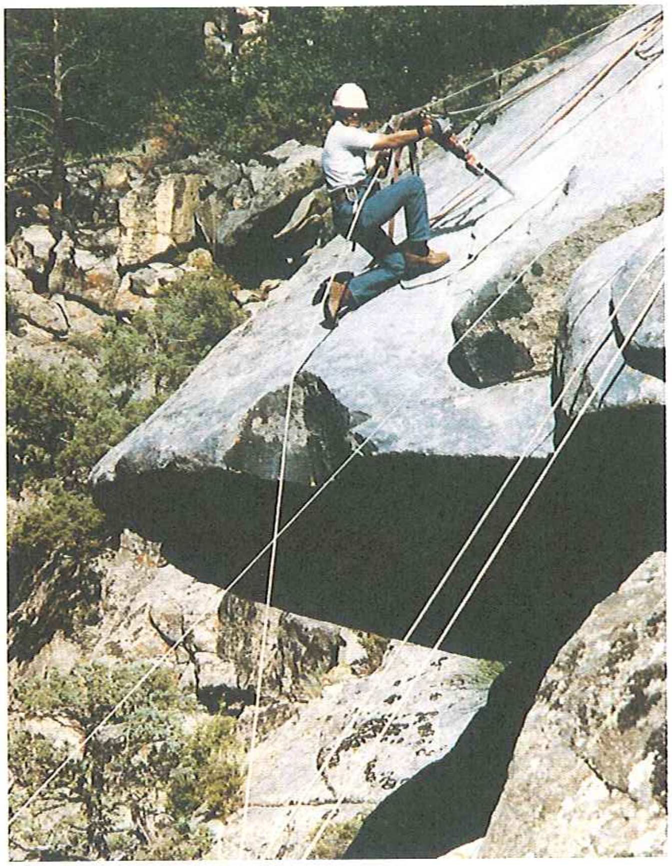


Figure 10-34. Drilling holes for the rock bars *(Courtesy Colorado Department of Transportation).*

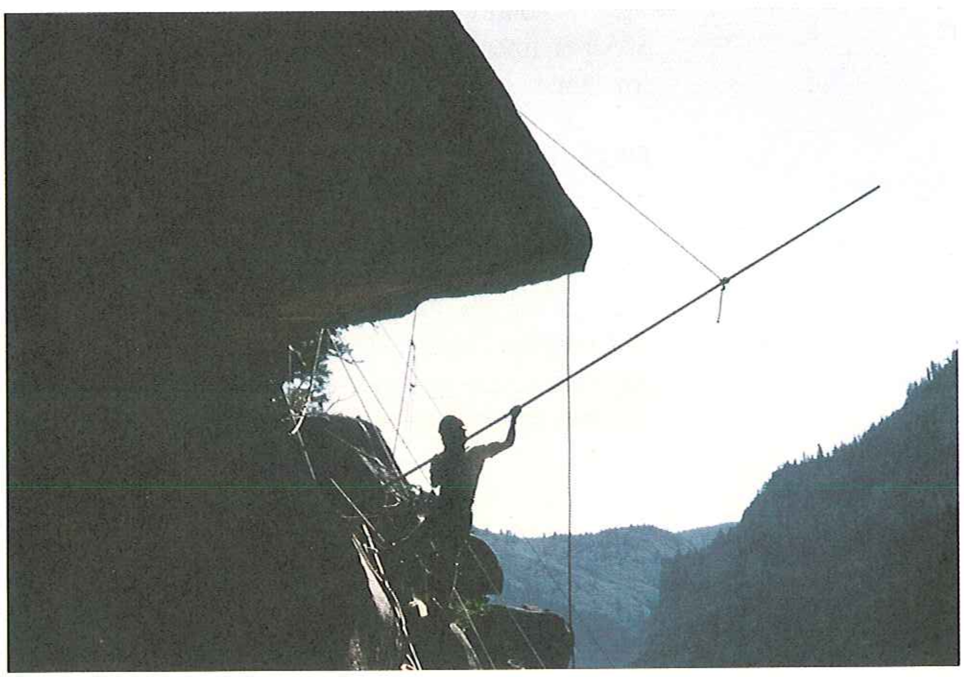


Figure 10-35. Placing the Dywidag bar into the drilled hole. The bottom 5 feet was grouted to develop the anchor after which the bar was tensioned. A watertight sleeve was placed over the ungrouted portion of the bar *(Courtesy Colorado Department of Transportation).*

The location of the lower joint was obvious during the drilling. Celtite slow set epoxy resin cartridges were used for the bond length only. It was not possible for workers to spin a 20-foot (6 meter) bar through 20-feet (6 meters) of epoxy cartridges by hand while on ropes and using a hand drill.

A three-man crew was required to install the rock bolts. The hole for the free bar length was left open so that more than half of the bar could be placed into the hole (figure 10-35) before the resistance of the resin stopped it. The bar was pushed through the resin by hand until about 5 to 6 feet (1.5 to 1.8 meters) still stuck out of the hole. From that point the hand drill was used to spin the bar to full depth.

To provide additional corrosion protection, greased PVC sheaths were slid down the bars the full free length and then pushed into the still soft resin to form a water-tight seal. A 100-ton (90.7Mg) calibrated stressing ram was used the following day to tension the bars to the specified load. A 5-minute proof test was completed on each bolt. Figure 10-36 shows three bar installations completed.

Total cost for the rock stabilization was $26,000 or $85 per lineal foot of bolt. Eighty per cent of this cost was for labor.

10.11. NORTH CASCADES HIGHWAY, WASHINGTON

On August 16, 1989, a large rockfall occurred at MP 124 on the North Cascades Highway, State Route 10, in Washington (figure 10-37). The rockfall was immediately adjacent to the east portal of Tunnel No. 2, which is located approximately 4 miles east of Newhalem, Washington.

The rockfall, which occurred at approximately 3:00 p.m., covered both the west- and east-bound lanes with approximately 1,000 yd^3 ($765m^3$) of rock debris and involved a 120 to 150 foot (36.6 to 45.8 meter) high rock slope. The failure (plane type) was controlled by two major discontinuities (figure 10-38). The first being a steeply dipping discontinuity paralleling the existing rock slope and dipping out of the slope at between 70 to 75 degrees. The second discontinuity dipped obliquely out of the slope at approximately 40 degrees and formed the release portion of the failure. Reports shortly after the failure indicated that a lot of water was running from this low-angle discontinuity.

Investigation--A field review of this area the following day indicated that only the bottom one-third of the kinematically controlled rock mass had actually failed leaving approximately 1500 to 2000 yd^3 (1147.5 to 1530m^3) of unstable rock material remaining on the slope. Several rock slope stabilization options were considered initially, including, 1) do nothing; 2) rock bolt the unstable rock mass; and 3) remove the unstable rock mass by trimming. Due to safety issues of working below the unstable rock mass and time constraints to open the roadway, it was decided to pursue the third option. However, the potential damage to the existing concrete portal of Tunnel No. 2, and the potential for affecting a Seattle City Light power transmission tower located downslope of the highway, complicated this option (figure 10-39). Communications with Seattle City Light indicated that replacement costs could be as high as $1 million if the power transmission tower was damaged severely. Because of the potential high liability costs involved, the Washington Department of Transportation accepted all liability for any damage to the adjacent facilities.

Construction--Conceptually, the rock slope trimming would consist of a controlled blast line located approximately 10 feet (3.1 meters) behind the high angle failure surface and drilled full depth. Fourteen holes would be required with a spacing of 36 inches (914mm) on center. In order to drill this controlled blast line, the contractor, Wilder Construction, was required to lift an air track drill to the top of the slope with a 450-ton (408Ms) crane. Because of height, the drill was disassembled and lifted in pieces and then reassembled at the top of the slope. The drill was "tied off" to a series of rock bolts installed upslope of the controlled blast line. The mobilization of the drill and other equipment to the top of the slope took 5 days to complete. Drilling of the controlled blast holes began on August 26 and was completed on August 29.

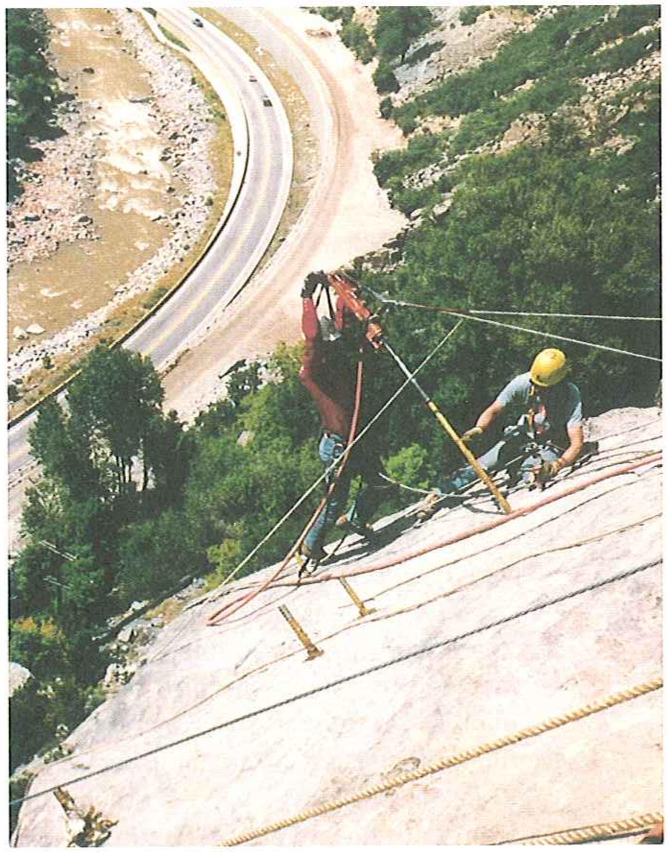


Figure 10-36. Several rock bars that have been installed, anchored, and tensioned *(Courtesy Colorado Department of Transportation).*

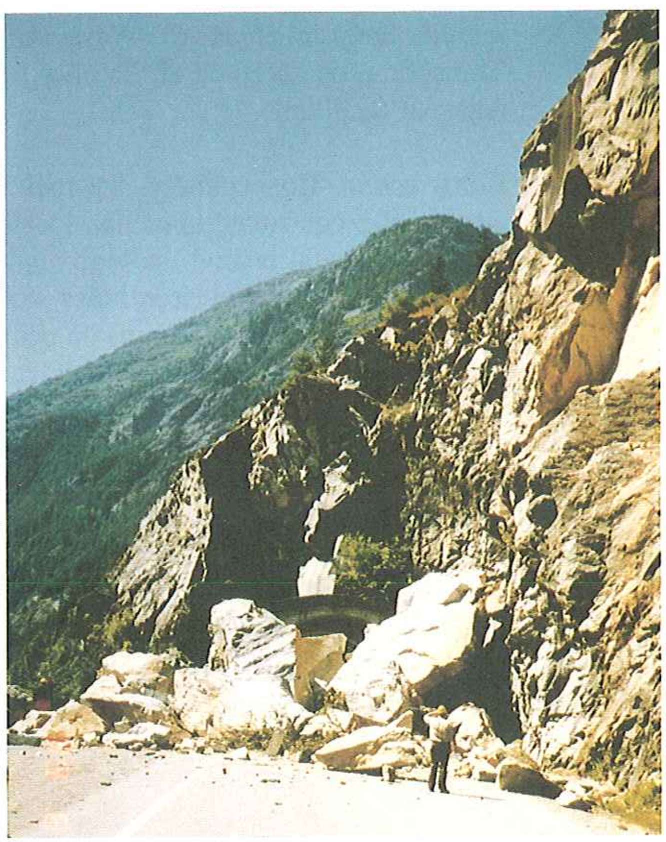


Figure 10-37. Large rockfall at the tunnel portal about 4 miles (26km) east of Newhalem, Washington completely blocks the highway *(Courtesy Washington Department of Transportation).*

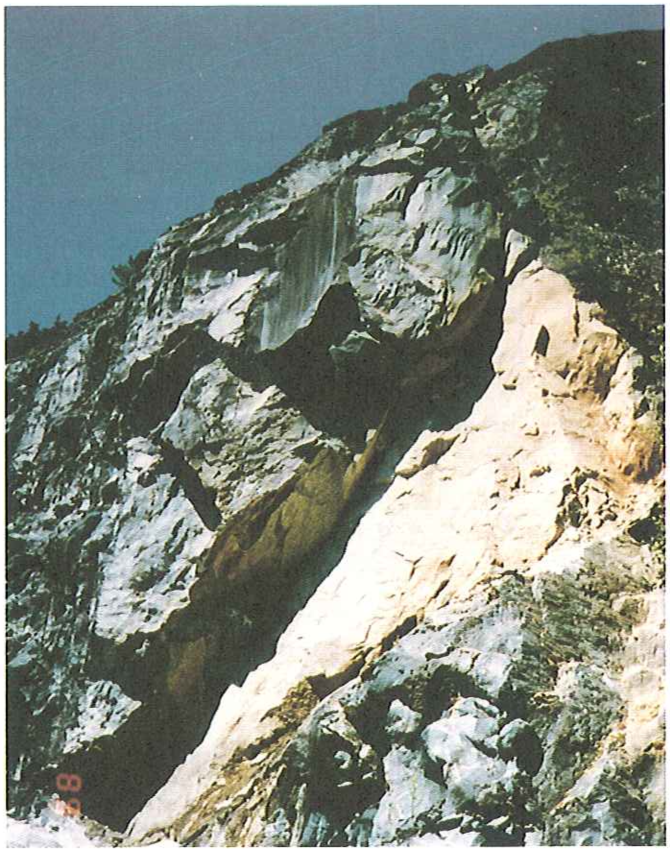


Figure 10-38. The rockfall was a planar failure sliding on a rough through-going joint that dipped out of the slope at about 70°. A release joint existed to the left *(Courtesy Washington Department of Transportation)*.

Figure 10-39. A major power transmission line had to be protected during the trim blast above the remaining rock, which was considered unstable *(Courtesy Washington Department of Transportation)*.

Hal Sheeran, a blasting specialist, was hired to assist the Washington Department of Transportation and the contractor to determine the explosive loads and delay sequence of the trim shot. The controlled blast holes explosive load were modified to include a heavy production load of explosive in the bottom 30 to 40 feet (9.2 to 12 meters) of the drill holes and conventional preshear explosives in the upper portions of the holes. The intent of the heavier explosive loads near the bottom was to "kick" the toe of the slope out, while the upper preshear explosives provided the hole-to-hole shear. The delay sequence of the shot started from the tunnel portal to the west and proceeded to the east toward the power transmission tower. In addition, the consultant recommended the construction of a large earthen deflection berm on the outside edge of the highway to protect the transmission line (figure 10-40).

The trim shot was made on August 30, 1992, 14 days after the initial rock slope failure (figure 10-41). The unstable rock mass was removed with no damage to either the east portal of the tunnel (figure 10-42) or the transmission tower. Review of sequential photographs of the blast indicated that the earthen deflection berm prevented much of the shot rock from impacting the transmission tower.

Removal of the rock debris on the highway required some secondary blasting to break the oversized material into manageable sizes. Within two days of the trim shot, the highway was opened to traffic. The work was conducted done under an emergency force account contract (cost plus fixed fee).

The closure of this highway received widespread attention in the press. Since this highway is a scenic highway only open during a portion of the year, nearby merchants who derive their livelihood from the tourists that travel the highway felt the economic impact of the work. Unfortunately, the rockfall occurred at the peak of the tourist season, which was only three weeks before the Labor Day weekend.

Figure 10-40. Catch and diversion dyke to protect the transmission line from blast rock *(Courtesy Washington Department of Transportation)*.

Figure 10-41. Trim blast detonated to remove the potentially unstable rock. No flyrock damaged the transmission line. The blast was designed specifically to minimize distant flyrock *(Courtesy Washington Department of Transportation)*.

Figure 10-42. Cleanup of the rockfall and trim blasted rock. The tunnel portal was not damaged *(Courtesy Washington Department of Transportation)*.

10.12. NORTH CASCADES HIGHWAY, WASHINGTON

This site is located at MP 125 on the North Cascades Highway, approximately 5 miles (8km) east of Newhalem, Washington. In the early 1980s a construction contract was awarded to improve the existing highway by removing a single-lane tunnel that was present at this location. As part of this project, the new slope, which was formed by the removal of the tunnel, was draped with a standard wire mesh slope protection consisting of vertical and horizontal cables with chain link fence material utilized as the slope protection.

Since the wire mesh slope protection was installed, the highest portion of the installation has failed twice; the last in 1989. The cause of the two wire mesh slope protection failures was large rockfalls generated from colluvial materials located high on the existing 260-foot (79 meter) high slope. In addition, mid-slope anchorage of the wire mesh slope protection contributed to the failures by restricting the downslope movement of the rockfall debris.

Design--Due to the previous unsuccessful wire mesh slope protection installations, it was decided to install much heavier slope protection using Brugg cable netting (figure 10-43). Washington State Department of Transportation's standard plan for wire mesh slope protection was modified to accommodate the Brugg cable net system.

- Horizontal spacing of the vertical support cable was decreased from 50 feet to 25 feet (15 meters to 7.6 meters) to accommodate the width of the Brugg cable net panels.

- Additional vertical support cable anchors were added at 25-foot (7.6 meter) spacing to increase the capacity of the cable net system.

- The vertical and horizontal support cables were increased from the standard 1/2-inch to 3/4-inch wire rope.

- Brugg 3/16-inch (11mm) wire rope net with a 8 inch by inch (203 by 203mm) cable net was specified for the cable net installation. The cable net would be laced to the vertical and horizontal support cables with 1/4 inch (1.6mm) wire rope.

- Macafferi double twist wire mesh fabric would be installed on top of the Brugg cable net to prevent smaller rock from going through the 8 inch by 8 inch (203 by 203mm) opening of the cable net.

Prior to the installation of the cable net system, hand scaling of the slope to remove loose unstable material was proposed.

Figure 10-43. Brugg cable netting being laid, unrolled, and readied for lifting into place *Courtesy Washington Department of Transportation.*

Construction--In June 1993, construction of the cable net system began with hand scaling loose unconsolidated colluvial material located at the crest of the existing slope. Because of the very dry nature of this material, the scaling time nearly doubled the original 90 crew hours set up in the contract. In addition, several large boulders were removed by drilling and shooting. Once the on-slope scaling had been completed, the additional cable anchors and the 3/4 inch (19mm) support cables were installed. With these support elements in place, the Brugg cable net was placed on the slope using a helicopter to lift multiple-preassembled panels (figure 10-44). As the Brugg cable net panels were placed, they were laced to the vertical and horizontal cable with 1/4 inch (1.6mm) wire rope (figure 10-45). Once Brugg cable nets had been placed on the slope, the Macafferi wire mesh fabric was installed and attached to the cable net with metal hog rings. The completed installation is shown in figure 10-46.

Construction time for this 59,400 ft^2 (5,524m^2) cable net installation was approximately three months.

Figure 10-44. Helicopter lifting the Brugg cable preassembled panels to be connected to the support cables *(Courtesy Washington Department of Transportation)*.

Figure 10-45. Brugg cable net installed and tied to wire rope *(Courtesy Washington Department of Transportation)*.

Figure 10-46. Completed cable net installation hanging over the slope *(Courtesy Washington Department of Transportation)*.

10.13. SPIRIT LAKE MEMORIAL HIGHWAY, WAHSINGTON

Site Conditions--This site is located along the Spirit Lake Memorial Highway, approximately 37 miles (59.6km) east of Interstate 5. During the construction of this segment of new highway, a large rock failure occurred in spring 1992, that involved approximately 300 feet (91.5 meters) of an existing 50 to 60 foot (15.3 to 18 meter) high rock slope. The failure (planar type) was controlled by a low angle (38°) discontinuity trending near parallel to the slope and daylighting near the base of the new slope (figure 10-47). A series of near-vertical discontinuities, also trending near parallel to the slope, were present. A detailed field review of this failure indicated that these near-vertical discontinuities were, in all likelihood, filled with water prior to failure.

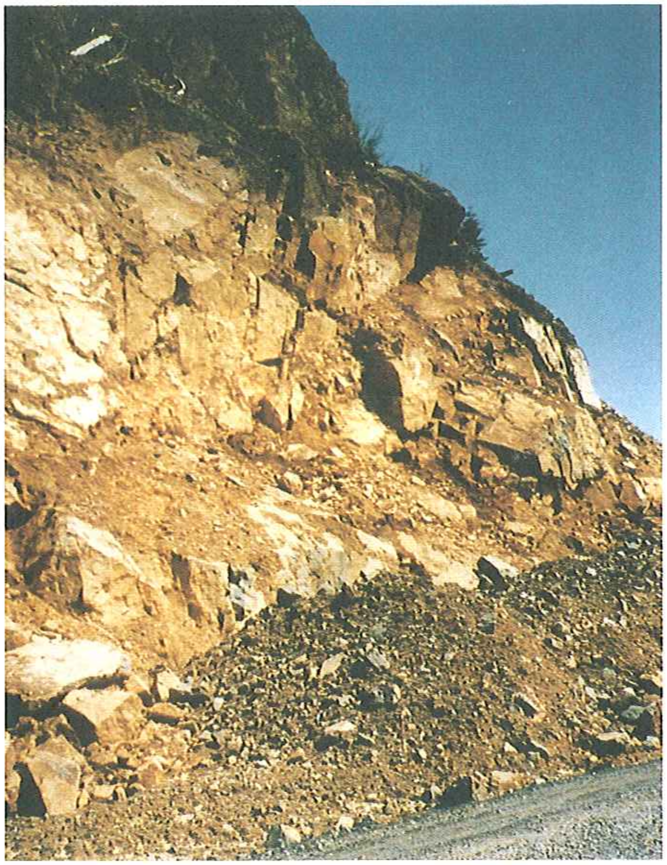


Figure 10-47. The rockfall slid down planar joint dipping about 38° toward the highway. A near vertical joint existed behind the rockfall zone *(Courtesy Washington Department of Transportation).*

Design--Because of the steepness of the existing ground above the slope failure, and the presence of a retaining wall-supported roadway embankment, options for correcting this slope failure were limited to the stabilization of the failed slope. Golder Associates of Redmond, Washington, in collaboration with Washington State Department of Transportation geotechnical staff, developed a stabilization program for this rock slope. The stabilization program included rock bolting unstable rock blocks that were present within the slope, the installation of horizontal rock drains to reduce the potential for buildup of excessive hydrostatic water pressures within the slope and the construction of a concrete buttress to stabilize a large unstable rock block near the base of the slope. The stabilization program was detailed on a series of color photo mosaics for use by the contractor and the field geotechnical personnel. Estimated quantities of post tensioned rock bolts, horizontal rock drains, and materials required for the construction of the concrete buttress were also provided. Rock bolts were designed for 25 kip post tensioned load, with proof testing of each bolt up to 120 percent of the design load utilizing a calibrated hydraulic jack. The rock bolts selected for use in this application were Dywidag 1-inch (25.4mm) threadbars utilizing Celtite polyester resin for anchorage. Bolt lengths were estimated at 20 feet (6 meters) initially.

Construction--Stabilization of this slope began in June 1992. Because of the height of the slope, all stabilization work was performed from a crane-supported platform and working from the top of the slope down (figure 10-48). As the drilling for the installation of the rock bolts progressed it became evident the rock quality behind the existing slope face was highly variable. To provide for an adequate bond length for the rock bolts, field criteria were established to guide the geotechnical field personnel in determining the adequacy of the drill hole for a rock bolt. Based on this field criteria, nearly half of the rock bolts installed were lengthened from 20 to 30 feet (6 to 9 meters). A total of 3300 linear feet of post tensioned grouted rock bolts were utilized in the stabilization of this slope (figure 10-49). In addition, 525 linear feet of horizontal drains were installed.

Once the upper portion of the rock slope had been stabilized, the large unstable rock block near the base of the slope was temporarily stabilized with eight 30-foot-long (9 meter) rock bolts. After completing this temporary work, the rock debris from beneath this rock block was removed so that the construction of the concrete buttress could begin. A series of short 3-foot-long (.9 meter) rock dowels grouted on 2-foot centers were installed to provide a positive connection between the rock and the concrete buttress and to support the rebar reinforcement grid (figure 10-50). Once these elements were completed, concrete forms were placed and 120 yd^3 ($92m^3$) of concrete required for the buttress was poured. After the concrete had cured, two rows of five rock bolts were installed through the buttress (figure 10-51).

The stabilization of this rock slope was completed in late August 1992. The work was complted under the force account provisions of the construction contract for this segment of the State Route 504 highway alignment.

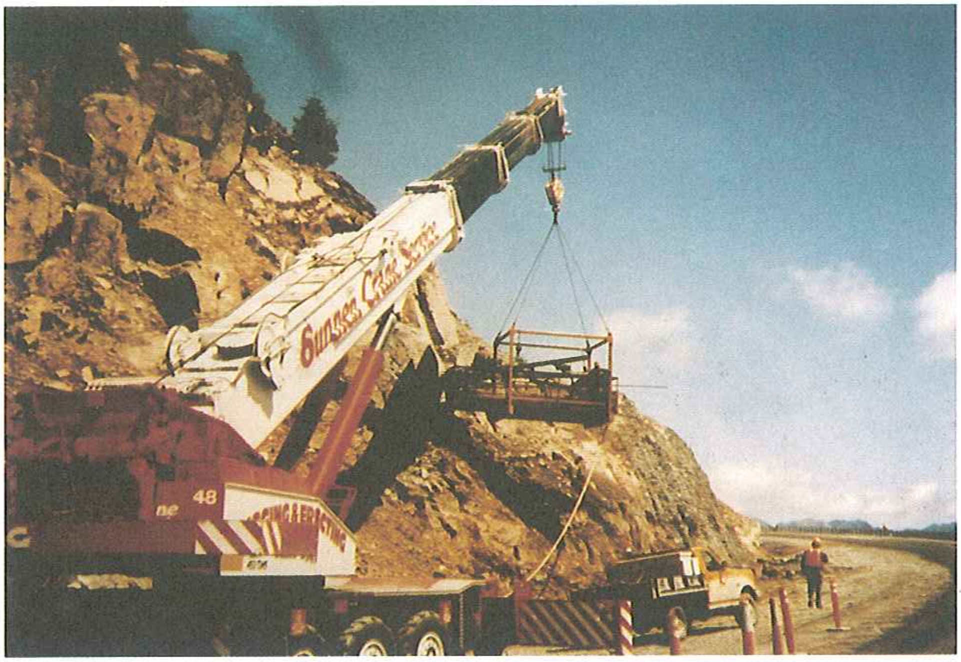


Figure 10-48. Work was performed from a large telescoping crane with a work platform *(Courtesy Washington Department of Transportation).*

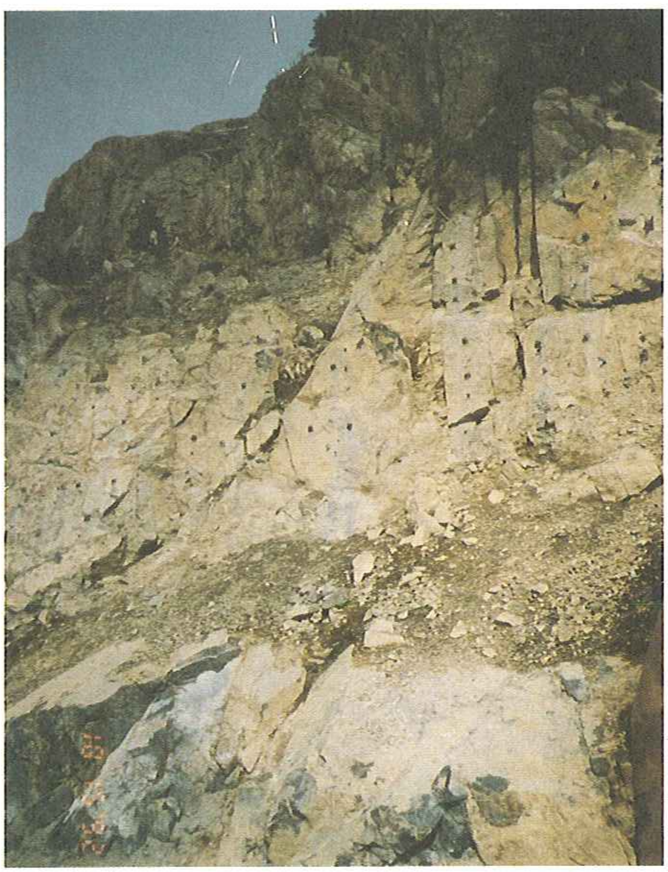


Figure 10-49. Steep blocks stabilized with 20 to 30-foot-long (6 to 9 meter) Dywidag rock bolts *(Courtesy Washington Department of Transportation).*

Figure 10-50. Grouted dowels spaced on 2 to 3 foot (.6 to .9 meter) centers to tie the buttress to the rock *(Courtesy Washington Department of Transportation)*.

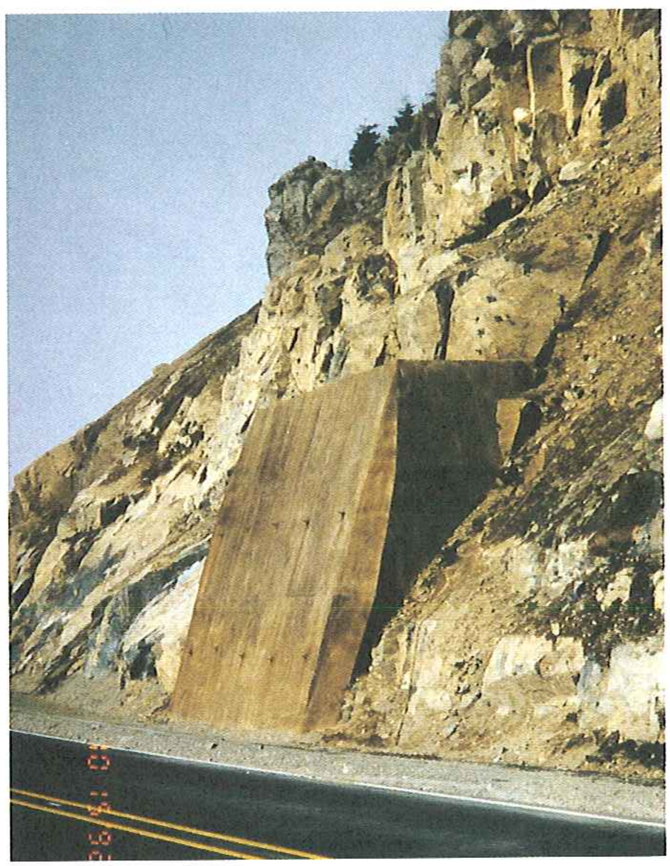


Figure 10-51. Concrete buttress completed at the toe of the slope. The buttress was tied to the rock with dowels and rock bolts *(Courtesy Washington Department of Transportation)*.

CHAPTER 11

ROCKFALL MITIGATION COSTS

The cost of rockfall mitigation and stabilization is very site-specific since the conditions that influence rock slope stability are extremely variable.

Typical factors that will influence the costs include:

- Use of State maintenance staff vs a contractor.
- Size and location of project.
- Availability of competent contractors.
- Contractor's wage structure--union vs nonunion.
- Topography--slope height, slope angle, access.
- Geology--rock and slope conditions.
- Climatic conditions.
- Traffic volume and controls required.
- Work space available at grade level.
- Detour potential.
- Productive hours per day.
- Construction control--noise, vibration, flyrock.
- Specialty equipment required--crane, helicopter.
- Environmental requirements--appearance, no rock in the river.

State officials or engineers also may have specific preferences regarding mitigation or stabilization procedures. For example, they may prefer passive dowels to rock bolts, or deep, wide ditches may be preferred to catch ditches, plus walls or fences.

Because of the above conditions, the costs of rockfall mitigation or stabilization may vary greatly from location to location and from State to State.

In order to provide information to assist in cost estimating throughout the country, California, Colorado, New York, Oregon, and Washington were requested to provide costs on recent projects for various mitigation and stabilization procedures. These costs have been assembled into a summary table for each State (tables 11-1 to 11-5). Since conditions vary considerably, costs were requested for both the low and high range.

It is emphasized that the cost data base is reasonably limited for most of the stabilization procedures. The data summarized therefore must be used with caution. As more experience develops, the costs can be refined.

There are blanks for some of the items. In these cases, the State has insufficient or no experience to supply costs.

As more rock mitigation projects are performed, it is recommended that data will be provided to FWHA so experience and cost data can be updated to enlarge the database.

Table 11-1. CALIFORNIA - COST SUMMARY - ROCKFALL MITIGATION					
Work Item	**Personnel, Equipment and Materials**	**Unit Rate**	**Low Range (dollars)**	**High Range**	**Comments**
Scaling	Foreman, 3 scalers front end loader, and operator	- Hour - Force Account - Per yd^3	200 50/person 200	300 100 400	Includes all scaling materials and supplies.
	Trucks	- Hour	50	65	To move rock.
	Crane plus operator	- Hour	75 for small 100 ft, 15 ton	300 for large 300 ft., 90 ton	For drag scaling or to lift a basket for scalers.
Trim Blasting	All personnel, equipment, blasting, rock removal	- Blaster - hr. - Drill - hr.	70 60/hr.	100 80/hr	When separate from scaling.
Rock Excavation to Widen - Deepen Ditches	All personnel, equipment, blasting and excavation	- per cubic yard (based on a min. of 2 to 3000 yd^3.)	15 No blasting. Rippable.	65 drilling and blasting	
Dowels Type 1 (through the unstable rock)	All personnel, materials, equipment and installation	- per 8 ft. dowel - per 12 ft. dowel	500 1000	800 1500	Include all drilling related costs and load proof testing.
Dowels Type 2 (at toe of rock)	All personnel, materials, equipment and installation	- per dowel	300	600	Include all drilling, concrete packing around the dowel.
Rock Bolts	All personnel, materials, equipment and installation	- per lineal foot (Williams hollow core, 1 1/2")	30	80	Include load proof tests.
Cable Lashing	All personnel, materials, equipment and installation	- Hour - Force Account	180 $50/hr/man 15/hr for drill & comp	300	
Shotcrete	All personnel, materials, equipment and installation	- Cubic yard Including reinforcing hanging on 10' centers	250 regular shotcrete	1000 w/steel fibers	Include all setup, admixtures, testing application, cleaning.
Horizontal Drains	All personnel, materials, equipment and installation	- per lineal foot installed - Perforated pipe	11.50		Drains not longer than 30 feet (6m).
Cable Reinforced Rock Catchment Fence	All personnel, materials, equipment and installation	- Lineal feet of fencing Specify height chain link fence	22 8/ft.	38 14/ft.	Include foundations.
Brugg Rock Net	All personnel, materials, equipment and installation	- per ft^2 of rock net	22 36% is for foundations	35 40% is for foundations	Include anchor cables and foundations.
Draped Mesh	Hexagonal 12 gage mesh w/cables every 40 ft.	- per ft^2	1.10 1.37 with color	2.00	Includes everything.
Buttresses	All personnel, equipment, materials and construction	- Square ft. of face	40 of face	75 of face	Include all site preparation.

Table 11-2. COLORADO - COST SUMMARY - ROCKFALL MITIGATION					
Work Item	**Personnel, Equipment and Materials**	**Unit Rate**	**Low Range**	**High Range**	**Comments**
Scaling	Foreman, 3 scalers frontend loader, foreman as operator	- Hour - Force Account (% Markup)			Includes all scaling materials and supplies.
	Trucks (Dump)	- Hour	46	50	To move rock.
	Crane plus operator 100 - 140 ton 200 ft. boom	- Hour	150	225	For drag scaling or to lift a basket for scalers.
Trim Blasting	All personnel, equipment, drilling, blasting and rock removal	- per cu. yd.	10	25	When separate from scaling.
Rock Excavation to Widen - Deepen Ditches	All personnel, equipment, blasting and excavation	- per cubic yard	6	15	
Dowels Type 1 (through the unstable rock)	All personnel, materials, equipment and installation	- per 12 foot dowel/foot	12	74	Include all drilling related costs.
Dowels Type 2 (at toe of rock)	All personnel, materials, equipment and installation	- Dowel			Include all drilling related costs and concrete packing around the dowel.
	Crane plus operator (as above)	- Hour	150	225	To hold basket to drill and install dowels.
Rock Bolts or Anchors	All personnel, materials, equipment and installation	- per lineal foot	23.50	65	Include load proof testing.
Cable Lashing	All personnel, materials, equipment and installation	- Total cost	9000	12000	Include anchors and holes.
Shotcrete	All personnel, materials, equipment and application	- Cubic yard	400	550	Include all preparation, admixtures, application, testing and cleanup.
Horizontal Drains	All personnel, materials, equipment and installation	- per lineal foot installed - No perforated pipe - Perforated pipe	9	20	10 - 100 ft. long
Cable Reinforced Rock Catchment Fence	All personnel, materials, equipment and installation Helicopter	- Lineal feet of fencing hr.	350		Include foundations 11 feet high
Brugg Rock Net or equivalent	All personnel, materials, equipment and installation	- per sq. ft. of rock net	34		Include anchor cables and foundation.
Buttresses	All personnel, equipment, materials and construction	- Total	10,000 forms % cu.yd.		Include all site preparation, forms and anchors.

Table 11-3. NEW YORK - COST SUMMARY - ROCKFALL MITIGATION					
Work Item	**Personnel, Equipment and Materials**	**Unit Rate**	**Low Range**	**High Range**	**Comments**
Scaling	Foreman, 3 scalers frontend loader, foreman as operator	- yd³ - With blasting	50 75	80 100	Includes all scaling materials and supplies.
	Trucks	- Hour			To move rock.
	Crane plus operator	- Hour			For drag scaling or to lift a basket for scalers.
Trim Blasting	All personnel, equipment, drilling, blasting and rock removal	- yd³ bid as unclassified examination with presplitting.	35	50	When separate from scaling.
Rock Excavation to Widen - Deepen Ditches	All personnel, equipment, blasting and excavation	- per cubic yard			
<u>Dowels</u> Type 1 (through the unstable rock)	All personnel, materials, equipment and installation	- per lineal foot	50	100	Include all drilling related costs.
<u>Dowels</u> Type 2 (at toe of rock)	All personnel, materials, equipment and installation	- per lineal foot	50	100	Include all drilling related costs and concrete packing around the dowel.
	Crane plus operator	- Hour			To hold basket to drill and install dowels.
Rock Bolts	All personnel, materials, equipment and installation	- per lineal foot	50	100	Include load proof testing.
Cable Lashing	All personnel, materials, equipment and installation	- Hour - Force Account (% Markup)			
Shotcrete	All personnel, materials, equipment and application	- Cubic yard			Include all preparation, admixtures, application, testing and cleanup.
Horizontal Drains	All personnel, materials, equipment and installation	- per lineal foot installed - No perforated pipe - Perforated pipe			Consider drains <u>not</u> longer than 30 feet.
Cable Reinforced Rock Catchment Fence	All personnel, materials, equipment and installation	- Lineal feet - Specify height	10 6 ft.		Include foundations.
Brugg Rock Net or equivalent	All personnel, materials, equipment and installation	- per sq. ft. of rock net	34		Include anchor cables and foundation.
Buttresses	All personnel, equipment, materials and construction	- Force Account (% Markup) - Cubic Yard			Include all site preparation.

Table 11-4. OREGON - COST SUMMARY - ROCKFALL MITIGATION					
Work Item	**Personnel, Equipment and Materials**	**Unit Rate**	**Low Range**	**High Range**	**Comments**
Scaling	Foreman, 2 scalers frontend loader, foreman as operator	- Hour - Force Account (% Markup)	140	200	Includes all scaling materials and supplies.
	Trucks	- Hour	40	60	To move rock.
	Crane plus operator	- Hour	125	250	For drag scaling or to lift a basket for scalers.
Trim Blasting	All personnel, equipment, drilling, blasting and rock removal				When separate from scaling. Blast scaling.
Rock Excavation to Widen - Deepen Ditches	All personnel, equipment, blasting and excavation	- per cubic yard	2.25	7	Add controlled blasting of 1.00 - 2.50 per lineal foot.
<u>Dowels</u> Type 1 (through the unstable rock)	All personnel, materials, equipment and installation	- per 12 foot dowel/foot	20	75	Include all drilling related costs. Include load proof testing.
<u>Dowels</u> Type 2 (at toe of rock)	All personnel, materials, equipment and installation	- per dowel			Include all drilling related costs and concrete packing around the dowel.
Rock Bolts	All personnel, materials, equipment and installation	- per lineal foot	20	75	Include load proof testing.
Cable Lashing	All personnel, materials, equipment and installation	- Each	1500	3000	Include 2 anchor bolts.
Shotcrete	All personnel, materials, equipment and application	- per sq. foot 3 inch thick	2.85	7.40	Include all preparation, admixtures, application, testing and cleanup.
Horizontal Drains	All personnel, materials, equipment and installation	- per lineal foot installed - Perforated pipe	15	23	Drains <u>not</u> longer than 30 feet.
Cable Reinforced Rock Catchment Fence	All personnel, materials, equipment and installation	- per sq. foot of fencing Specify height	1.25	2.00	Include foundations.
Brugg Rock Net or equivalent	All personnel, materials, equipment and installation	- per lineal ft. of rock net - 5 - 8 ft. high	50	200	Include anchor cables and foundation.
Buttresses	All personnel, equipment, materials and construction	- Force Account (% Markup) - Cubic Yard	10	15	Include all site preparation.
Slope Screening	All personnel, equipment, materials & construction	- per sq. ft.	1.00	1.50	Include hardware and anchors.

Table 11-5. WASHINGTON - COST SUMMARY - ROCKFALL MITIGATION					
Work Item	**Personnel, Equipment and Materials**	**Unit Rate**	**Low Range**	**High Range**	**Comments**
Scaling	Foreman, 2 scalers frontend loader, foreman as operator	- Hour - Force Account (% Markup)	100	200	Includes all scaling materials and supplies.
	Trucks	- Hour	-	-	To move rock.
	Crane plus operator	- Hour	220	265	For drag scaling or to lift a basket for scalers.
Trim Blasting	All personnel, equipment, drilling, blasting and rock removal	- per cubic yard	92		When separate from scaling. Blast scaling.
Rock Excavation to Widen - Deepen Ditches	All personnel, equipment, blasting and excavation	- per cubic yard	-	-	
Dowels Type 1 (through the unstable rock)	All personnel, materials, equipment and installation	- per 8 foot dowel - per 12 foot dowel	80 120	280 420	Include all drilling related costs. Include load proof testing.
Dowels Type 2 (at toe of rock)	All personnel, materials, equipment and installation	- per dowel	75	-	Include all drilling related costs and concrete packing around the dowel.
	Crane plus operator	- Hour	220	265	To hold basket to drill and install dowels.
Rock Bolts	All personnel, materials, equipment and installation	- per lineal foot	25	200	Include load proof testing.
Cable Lashing	All personnel, materials, equipment and installation	- Each	-	-	
Shotcrete	All personnel, materials, equipment and application	- per sq. foot	6.00	-	Include all preparation, admixtures, application, testing and cleanup.
Horizontal Drains	All personnel, materials, equipment and installation	- per lineal foot installed - Perforated pipe	6.00	8.00	Drains not longer than 30 feet.
Cable Reinforced Rock Catchment Fence	All personnel, materials, equipment and installation	- per sq. foot of fencing Specify height	-	-	Include foundations.
Brugg Rock Net or equivalent	All personnel, materials, equipment and installation	- per sq. ft. of rock net	5.00	7.50	Include anchor cables and foundation.
Buttresses	All personnel, equipment, materials and construction	- Force Account (% Markup) - Cubic Yard	258.00		Include all site preparation. Includes anchors.

CHAPTER 12

LIABILITY CONSIDERATIONS

12.1. INTRODUCTION

There have been many instances on United States highway systems where accidents, injuries, and death have resulted from rockfalls and slides. The most common accident is one where the vehicle runs into a rockfall on the highway. This usually occurs where sight distance is limited and the vehicle cannot stop in time. The second most frequent type of accident is where the rockfalls on or into the vehicle. In some instances, an accident occurs when the driver swerves to miss the rockfall and looses control of the vehicle or runs into a second vehicle.

In all instances today where accidents result from rockfall, the likelihood of litigation against the State is very high.

12.2. STATE LIABILITY EXISTING HIGHWAYS

To outline the liability issues for rockfall on existing highways, the legal departments of four States were contacted to advise on the law and practice related to rockfall liability in their States. It will be noted there are some discrete differences in the law and interpretation in different States.

What follows are the legal and liability considerations for these States.

12.3. STATE OF WASHINGTON

There is no State legal policy, as such, regarding rockfall liability. Washington waived sovereign immunity in 1961 and the State department of transportation is liable to motorists if failure to exercise ordinary care in highway design, maintenance, or signing leads to an injury. The standard jury instruction on the liability of Washington road authorities is enclosed (WPI-140.01). Whether there is liability in a particular case is very fact specific. It depends on highway age and construction standards, accident history, magnitude of the program, cost, and feasibility of correction, and other factors.

Analysis of Instructions

WPI 140.01

SIDEWALKS, STREETS AND ROADS—DUTY OF MUNICIPALITY

A [county] [city] [town] [state] has a duty to exercise ordinary care in the [construction] [design] [maintenance] [repair] of its public [roads] [streets] [sidewalks] to [keep] [construct] them in a [manner] [condition] that is reasonably safe for ordinary travel.

NOTE ON USE

Use bracketed material as applicable. If there is an issue of contributory negligence use WPI 140.01.01, Sidewalks, Streets and Roads—Contributory Negligence, with this instruction. This instruction covers the general duty only. If there are specific standards or regulations applicable, they will have to be covered by specific instructions.

COMMENT

It is well established that a municipality has a duty to exercise ordinary care to keep its highways reasonably safe for ordinary travel. Boeing Co. v. State, 89 Wn.2d 443, 572 P.2d 8 (1978); Owens v. Seattle, 49 Wn.2d 187, 299 P.2d 560, 61 A.L.R.2d 417 (1956); Parker v. Skagit County, 49 Wn.2d 33, 297 P.2d 620 (1956).

The qualification formerly in this instruction that public highways must be reasonably safe for "persons using them in a proper manner and exercising ordinary care for their own safety" was based on Argus v. Peter Kiewit Son's Co., 49 Wn.2d 853, 307 P.2d 261 (1957). The committee believes that this qualification no longer applies since contributory negligence is not now a bar to recovery. See the Comment to WPI 140.01.01, Sidewalks, Streets and Roads—Contributory Negligence.

If there is an inherently dangerous or deceptive condition of the roadway, the duty of ordinary care may include the duty of erecting and maintaining proper warning signs when necessary. Provins v. Bevis, 70 Wn.2d 131, 422 P.2d 505 (1967); Ulve v. City of Raymond, 51 Wn.2d 241, 317 P.2d 908 (1957); Bradshaw v. City of Seattle, 43 Wn.2d 766, 264 P.2d 265, 42 A.L.R.2d 800 (1953). The general instruction should not single out any of the several available methods of discharging the duty of the municipality.

The duty of care extends to design as well as maintenance. Raybell v. State, 6 Wn.App. 795, 496 P.2d 559 (1972).

The duty of a municipality or a contractor working in the street is limited to keeping the street reasonably safe for ordinary travel of a kind that can reasonably be anticipated. There is no duty to anticipate that a pedestrian will ignore an available crosswalk and jay walk into a construction area in the street which has been made safe for vehicle traffic only. The duty of the municipality is independent of any question of contributory negligence of the pedestrian. The doctrine of comparative negligence does not enhance the duty of the defendant. Hansen v. Washington Natural Gas Co., 95 Wn.2d 773, 632 P.2d 504 (1981).

Fernandez v. Department of Highways, 49 Wn.App. 28, 741 P.2d 1010 (1987), holds that a pedestrian who uses a highway or bridge which is not open to pedestrian traffic is a trespasser and is owed only the duty not to be willfully or wantonly injured. Breivo v. Aberdeen, 15 Wn.App. 520, 550 P.2d 1164 (1976), rejected the argument that a municipality owes no duty to persons riding with careless drivers.

Library References:

C.J.S. Municipal Corporations § 944.
West's Key No. Digests, Municipal Corporations ⚷822(1).

Liability on an older highway would be judged on the basis of factors listed in the last sentence above. Generally, there is less liability on older roads if they met standards when built and the cost of remedial measures is high. However, a very bad accident history can change this conclusion.

"Design standard" is a factor but not a deciding factor. The State department of transportation cannot win solely on a design standard defense.

State department of transportation liability for rockfall is generally determined by state engineers and the attorney general's office who analyze the factors mentioned above and making a judgment concerning whether a jury would likely find liability. Settlement offers range from zero to very large, depending on specific facts.

Liability for new construction is determined under the same rules as for older construction, but there is more liability if new standards are higher and the Department of Transportation has fallen short of those standards. Higher standards for new construction reduce liability if the standards are satisfied and fewer accidents occur; higher standards increase liability if they are unrealistic and difficult to satisfy for budgeting or engineering reasons.

Warning signs are less significant for rockfall than for conditions such as curves, stop ahead signs, and road defects. Most danger from rockfall cannot be mitigated by signs. However, signs will lessen liability if their use can possibly reduce accident potential. Washington State has had some lawsuits alleging liability for failure to have warnings of possible fallen rock on the roadway.

Rockfall liability is a difficult problem in this State because Washington is a western state with high mountains and many freeze-thaw cycles. Most of the roads with rockfall problems were built long ago and problems are difficult to correct because of budget and environmental issues involving mountainous areas and highway relocation. As a matter of policy, the State should probably be given design immunity in this area because much of the liability is really a result of budget considerations on money available to rebuild or repair older highways--it is not a result of carelessness or negligence by the department of transportation engineers or maintenance forces.

12.4. STATE OF CALIFORNIA

State liability for accidents, injury or death on State highways resulting from rockfall is governed by the California Tort Claims Act (California Gov. Code, 815 et seq.). The State can be held liable if rocks on the road or a location that has rockfalls constitutes a dangerous condition of public property (Gov. Code, 835). A dangerous condition is generally one that presents a substantial risk of harm to persons using the property with due care (Gov. Code, 830, subd. (a)). Where feasible, the State uses slope protection devices or rockfall mitigation techniques in those areas that pose a substantial risk of harm to users of its property. Where it is not feasible, the risk posed by a dangerous condition can be mitigated with of a sign warning of the dangerous condition. In many ways, it comes down to whether or not the State acted reasonably under all the circumstances.

Generally, the State is not required to bring all its existing highways up to current design standards. The courts recognize there are limited resources to undertake such a project. However, where appropriate new construction should take into account rockfall mitigation solutions.

The State does have a responsibility to maintain its highways so that falling rocks and rocks on the road do not pose substantial risks to drivers. In California, maintenance forces do rock patrol on roads in mountainous areas during times when rocks may fall (*for example.*, rain, freeze-thaw conditions, wind). Periodically, the slopes may be scaled to dislodge loose rocks and clean up catch basins.

With regard to case law on the subject, refer to Van Alystyne, California Government Tort Practice (Cont. Ed. Bar 3d ed. 1992).

The Legal Division of the department of transportation has had several rockfall cases in the last decade. Experience has been that cases involving rocks that fall directly on vehicles are more difficult to defend than those involving drivers hitting rocks on the road. In the former, a warning sign does little to protect a driver from dodging a falling rock. In that situation, measures taken by the State to prevent a rock from falling in the first place or

§ 815. Liability for injuries generally; immunity of public entity; defenses

Except as otherwise provided by statute:

(a) A public entity is not liable for an injury, whether such injury arises out of an act or omission of the public entity or a public employee or any other person.

(b) The liability of a public entity established by this part (commencing with Section 814) is subject to any immunity of the public entity provided by statute, including this part, and is subject to any defenses that would be available to the public entity if it were a private person.

§ 830. Definitions

As used in this chapter:

(a) "Dangerous condition" means a condition of property that creates a substantial (as distinguished from a minor, trivial or insignificant) risk of injury when such property or adjacent property is used with due care in a manner in which it is reasonably foreseeable that it will be used.

(b) "Protect against" includes repairing, remedying or correcting a dangerous condition, providing safeguards against a dangerous condition, or warning of a dangerous condition.

(c) "Property of a public entity" and "public property" mean real or personal property owned or controlled by the public entity, but do not include easements, encroachments and other property that are located on the property of the public entity but are not owned or controlled by the public entity.

§ 830.2. Minor, trivial or insignificant nature of risk

A condition is not a dangerous condition within the meaning of this chapter if the trial or appellate court, viewing the evidence most favorably to the plaintiff, determines as a matter of law that the risk created by the condition was of such a minor, trivial or insignificant nature in view of the surrounding circumstances that no reasonable person would conclude that the condition created a substantial risk of injury when such property or adjacent property was used with due care in a manner in which it was reasonably foreseeable that it would be used.

§ 830.4. Failure to provide traffic control signals or signs

A condition is not a dangerous condition within the meaning of this chapter merely because of the failure to provide regulatory traffic control signals, stop signs, yield right-of-way signs, or speed restriction signs, as described by the Vehicle Code, or distinctive roadway markings as described in Section 21460 of the Vehicle Code.

§ 830.8. Failure to provide traffic or warning signals; exception

Neither a public entity nor a public employee is liable under this chapter for an injury caused by the failure to provide traffic or warning signals, signs, markings or devices described in the Vehicle Code. Nothing in this section exonerates a public entity or public employee from liability for injury proximately caused by such failure if a signal, sign, marking or device (other than one described in Section 830.4) was necessary to warn of a dangerous condition which endangered the safe movement of traffic and which would not be reasonably apparent to, and would not have been anticipated by, a person exercising due care.

§ 835. Conditions of liability

Except as provided by statute, a public entity is liable for injury caused by a dangerous condition of its property if the plaintiff establishes that the property was in a dangerous condition at the time of the injury, that the injury was proximately caused by the dangerous condition, that the dangerous condition created a reasonably foreseeable risk of the kind of injury which was incurred, and that either:

(a) A negligent or wrongful act or omission of an employee of the public entity within the scope of his employment created the dangerous condition; or

(b) The public entity had actual or constructive notice of the dangerous condition under Section 835.2 a sufficient time prior to the injury to have taken measures to protect against the dangerous condition.

to prevent the falling rock from reaching the road, are most important. The department has not been successful in convincing a jury that doing nothing but scraping the rocks off the road after they fall is a reasonable approach to the problem. That approach has been likened by plaintiffs' attorneys to "Russian roulette." In cases where a driver collides with a fallen rock, warning signs can effectively put responsibility on the driver to be extra alert for rocks on the road. A driver's failure to avoid such rocks has been held to be negligence.

12.5. STATE OF NORTH CAROLINA

Recovery by individuals for injuries and damages against the State of North Carolina is made pursuant to the provisions of the North Carolina Tort Claims Act,N.C.G.S. § 143-291 et seq. This requires a claim or action against the State department based on the negligence of specific officers, employees, agents or involuntary servants of the State department. The maximum award is presently $100,000 per claim with no limit on the number of claims per incident.

The legal policy of this State concerning accidents, injury or death as a result of rockfall on highways is the same as for any other accident, injury, or death. Using the framework of the Tort Claims Act, the State attempts to determine whether the negligence of department of transportation employees was a proximate cause of the rockfall either in the design, construction, or maintenance of the highway or road in question. The standard of care is that of the generally accepted standards of highway engineers in each of those areas and includes the standards set forth by the American Association of State Highway Officials and the *Manual on Uniform Traffic Control Devices*.

The age of the highway is a factor in each claim. If the issue is design, the standards at the time of design are applicable and are a deciding factor. Each case, however, will not be limited to the design but will also include issues of maintenance and construction that involve inspection and monitoring the highways. If State engineers have actual or constructive notice of a dangerous area of rock slides, they are required to take reasonable action to correct the problem. What is reasonable depends on many factors, including time and resources available.

Warning signs do have legal significance in North Carolina. Those signs are evidence of reasonable action by department of transportation employees and relevant evidence on the issue of contributory negligence, which is a legal issue provided for in our Tort Claims Act.

Although three cases are pending, there have been no successful cases against the North Carolina Department of Transportation as the result of rock slides on highways in the last 10 years.

12.6. STATE OF NEW YORK

The State of New York cites case decisions rather than referring to State Legal Codes. The results of recent cases are summarized below.

A. Cerasoli vs The State of New York.

The claimant's car was struck by small rocks. He stopped several minutes, protected himself with a blanket and crawled under the dashboard, then restarted the car and drove off. The rocks were the size of a softball or smaller. At the time he stopped the car, there were more than 10 rocks on the roadway. The claimant had driven the highway one to five times per week for 15 years. He observed rocks on the road and reported their presence to authorities. However, he had never had a prior rock-related accident and could not recall rocks on the highway at that location before the instant accident.

The claim was dismissed for several reasons.

- The State is not the insurer of its highways.

- The mere happening of an accident creates no presumption of liability against the State.

- It was incumbent upon the claimant to prove that the State had a notice of dangerous condition at the accident site. The claimant acknowledged that he never saw rocks on the highway at the location in question prior to his accident.

- Rock slide accidents can occur in the absence of someone's negligence.

- There was no evidence that the property adjacent to the accident site was owned by the State; the rocks causing the accident, cannot therefore be said to have been within the exclusive contract of the State.

<u>B.</u> <u>*Updike vs the State of New York*</u>

The claimant's car struck a rock or rocks in the roadway causing damage to the undercarriage of the vehicle. About 40 minutes before the accident, a local police officer had driven through the area where the rocks were on the highway and did not strike them. He radioed his dispatcher to call the NY State Department of Transportation. Cleanup of the rocks was completed between 8 and 10 a.m. that morning. A review of the evidence noted the State received notice of the problem not earlier than 8 minutes before the accident.

The Court found the State of New York free of negligence and dismissed the claim based on several issues.

- The State is not an insurer for the users of State highways.

- The mere happening of an accident does not establish negligence on the part of the State.

- The claimant has the burden of proving the State had actual or constructive knowledge of the dangerous situation. Constructive knowledge arises when the dangerous condition existed for so long a period of time that it should have been observed.

C. *Mapley vs the State of New York.*

The claimant suffered personal injuries when he lost control of a tractor trailer after it struck a rock in the eastbound lane of the NY State Thruway at Mile 213.8.

A "Fallen Rock Zone" was marked by signs every 1/4 mile (4km) for a length of 1 1/2 miles (2.4km). Prior rockfalls had occurred in the area. The rock slope design called for a 1H:3V slope in the rock, a 2-foot (.6 meter) shelf at the soil rock contact and a 2H:1V slope in overlying soil. The slope was not constructed according to the design. The reason given was to stay within the right of way and avoid the need to acquire extra property.

The claimant was familiar with the highway and the Fallen Rock Zone. The weather was dark and rainy and the vehicle was travelling 50 mi/h (80.5km/h) in a 55 mi/h (88.6 km/h) zone.

The claimant suffered severe facial cuts and a severe knee cut. He is permanently scarred in the face and lost some mobility to his knee. He was off work for 10 weeks.

The Court held the following:

- The State was held negligent in failing to construct the highway in accordance with its design and such negligence was a substantial factor in bringing about the claimant's accide nt and injuries. The faulty construction resulted in the slope being less stable than designed and increased the likelihood of a rockfall.

- The claimant was operating his vehicle at a speed that was too fast for the conditions and potential hazards then and there existing. The claimant was negligent and such negligence was a substantial factor in bringing about the accident and his injuries. Damages awarded are to be reduced 20 percent.

- Damages were fixed at $45,797.01, less 20 percent.

D. *Klein and Neier vs State of New York and New York State Thruway Authority*

The claimants were driving on the New York Thruway about 11 miles north of New York City. A very large boulder struck the car injuring the driver, killing the passenger, and destroying the car.

There had been four prior rock slides at the site. The Authority had described this location in an internal memorandum as "The most hazardous rock slope in the New York Division."

Strapping and bolting had been installed in 1975. A Thruway inspector stated he had inspected the slope 12 times in 13 years but no intervals were specified. He said he got out of the car and looked at the site but did not go up the slope or measure anything. No regular monitoring was performed. Specialist evidence was presented that slopes move before they fail and that this could be measured.

The Court held that the New York State Thruway Authority was negligent in not periodically inspecting the site since there was a past history of rockfall. That negligence was the prominent cause of the damages here and that the damages were foreseeable.

Damages of $972,474.40 were awarded.

E. Blodgett vs State of New York

The claimant was driving north on the Palisades Interstate Parkway at about 50 to 55 mi/h (80.5 to 88.6km/h). Adjacent to a steep rock cut, her windshield exploded and she was struck in the face by a rock about 10 inches (254mm) square. The principal impact was to the left side of her face. She has no recollection of the event other than her driving and the windshield broke.

The claimant suffered the loss of one eye, required major head surgery to repair fractures and lacerations followed by extensive plastic surgery including bone grafts.

A state trooper attended the accident. He had observed rocks on the highway previously and had stopped and removed them. He advised his headquarters. He also had observed State maintenance forces removing rocks or boulders from the shoulder of the road in the accident area.

There was evidence that the state engineers and maintenance staff knew accidents had happened. No one recalled any falling rock signs being located in the area.

A previous rockfall inspection stated that "The evidence indicates that falling rock has reached the pavement in the past and is likely to do so in the future."

Several requests for funding to stabilize the area were made during the years and rejected prior to the accident. Some 6 months before the accident, an appropriation was approved but the work could not be started prior to the accident.

The court held the following:

- The defendant failed to prove that this section of the parkway was properly signed to warn of the rockfall danger, a danger to which the State forces were well aware. This failure to warn of danger was negligence and a proximate cause of this accident.

- The State is under a legal duty to exercise reasonable care to guard against foreseeable dangers in the construction and operation of its highways; and, to maintain them in a reasonably safe condition for use by the travelling public. This duty extends not only to the roadway proper, but also to conditions adjacent to and above the roadway which could be reasonably expected to result in injury and damage.

- Under the evidence presented, the endeavour by the State to excuse its negligence by claiming governmental immunity is not credible and without merit.

- The State Department of Transportation should have made use of funds available in 1975 to correct the situation or should have used emergency funds to remedy the situation or in lieu thereof, should have closed the highway until it could be corrected.

The State was negligent and its negligence was the sole and foreseeable proximate cause of this accident and the severe and permanent injuries suffered by the claimant.

The claimant was awarded $1,022,990.

12.7. U.S. CODE FOR HIGHWAYS

The 1992 edition of the United States Code--Title 23 Highways includes Laws of the 102nd Congress, First Session (1991). It defines an update on safety for new highways and reconstruction of existing highways.

Significant extracts from the 1992 Code edition are as follows:

"Title 23 sets out the law relating to Highways...The numerous Federal-Aid Highway and Highway Safety Acts provide the latest policies and programs for highways within a framework of competing national and local interests, preferences and resources."

"Since the Interstate System is now in the final phase of completion it shall be the national policy that increased emphasis be placed on the construction and reconstruction of the other Federal-aid systems...in order to bring all of the Federal-aid systems up to standards and to increase the safety of these systems to the maximum extent."

"The term "highway safety improvement project" means a project which corrects or improves high hazard locations, eliminates roadside obstacles,..."

Section 109--Standards--states, "The Secretary shall not approve plans and specifications for proposed highway projects under this chapter if they fail to provide for a facility (1) that will adequately meet the existing and probable future traffic needs and conditions in a manner conducive to safety, durability and economy of maintenance; (2) that will be designed and constructed in accordance with standards best suited to accomplish the foregoing objectives and conform to the particular needs of each locality."

The result of this updated policy will be that liability regarding damage, injury or loss of life due to rockfall will be more rigorously applied by the courts. This will require States to pay additional attention to rock slope inspection, evaluation, and rockfall mitigation.

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 1

Describe the unstable conditions:____________________________

List the type(s) of failure:____________________________

List the cause(s) of failure:____________________________

Recommend the stabilization program:____________________________

List traffic control requirements during stabilization:____________________________

List specifications to include in the contract:____________________________

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 2

Describe the unstable conditions:______________________________

List the type(s) of failure:______________________________

List the cause(s) of failure:______________________________

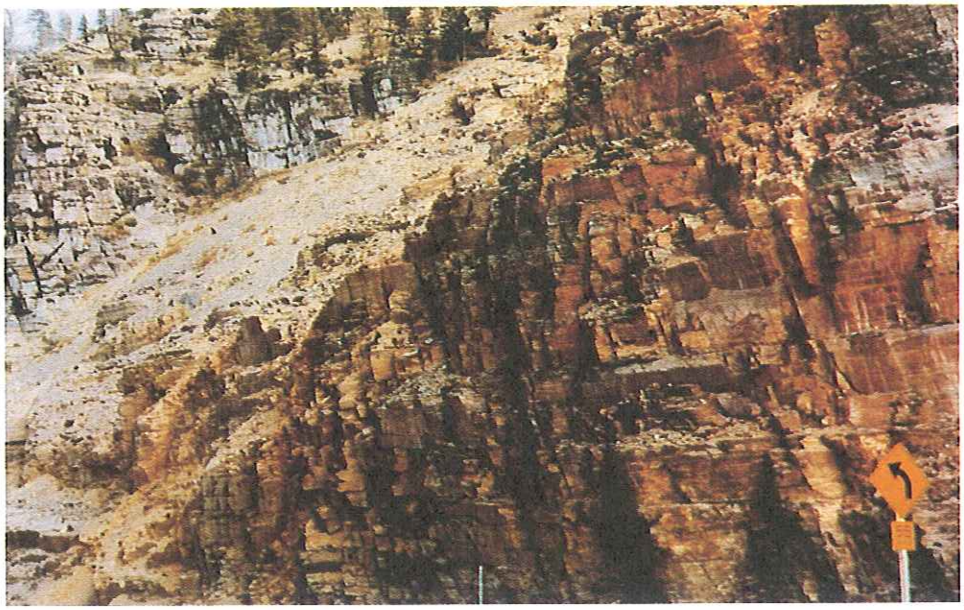
Recommend the stabilization program:______________________________

List traffic control requirements during stabilization:______________________________

List specifications to include in the contract:______________________________

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 3



Describe the unstable conditions:____

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List the type(s) of failure:____

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List the cause(s) of failure:____

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Recommend the stabilization program:____

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List traffic control requirements during stabilization:______________________

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List specifications to include in the contract:_______________________________

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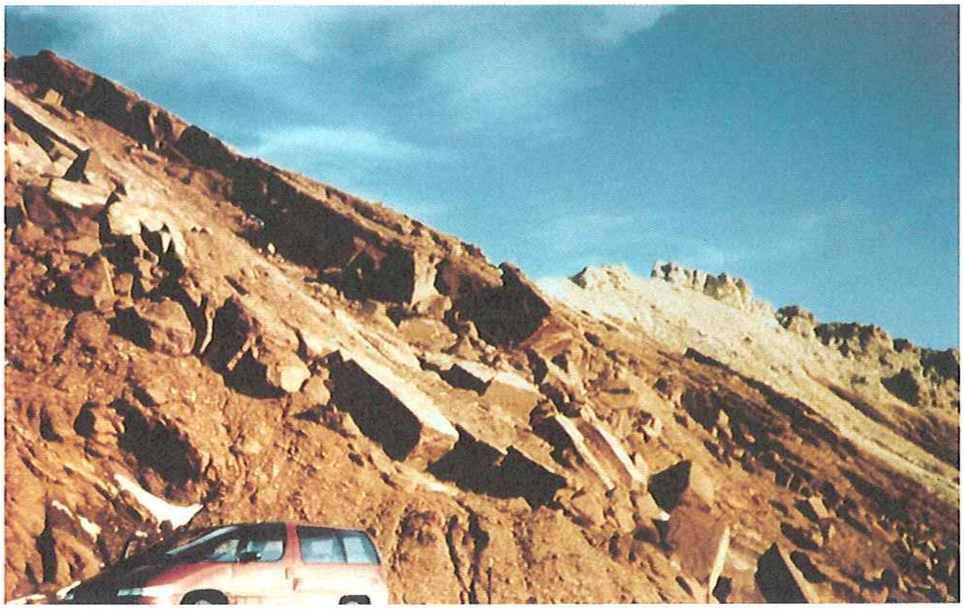
MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 4



Describe the unstable conditions:

List the type(s) of failure:

List the cause(s) of failure:

Recommend the stabilization program:

List traffic control requirements during stabilization:

List specifications to include in the contract:

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 5

Describe the unstable conditions:

List the type(s) of failure:

List the cause(s) of failure:

Recommend the stabilization program:

List traffic control requirements during stabilization:

List specifications to include in the contract:

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 6

Describe the unstable conditions:________

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List the type(s) of failure:______________

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List the cause(s) of failure:_____________

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Recommend the stabilization program:______________________________________

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List traffic control requirements during stabilization:_________________________

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List specifications to include in the contract:_______________________________

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MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 7

Describe the unstable conditions:____

List the type(s) of failure:____

List the cause(s) of failure:____

Recommend the stabilization program:____

List traffic control requirements during stabilization:____

List specifications to include in the contract:____

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 8

Describe the unstable conditions:______________________________

List the type(s) of failure:______________________________

List the cause(s) of failure:______________________________

Recommend the stabilization program:______________________________

List traffic control requirements during stabilization:______________________________

List specifications to include in the contract:______________________________

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 9

Describe the unstable conditions:____

List the type(s) of failure:____

List the cause(s) of failure:____

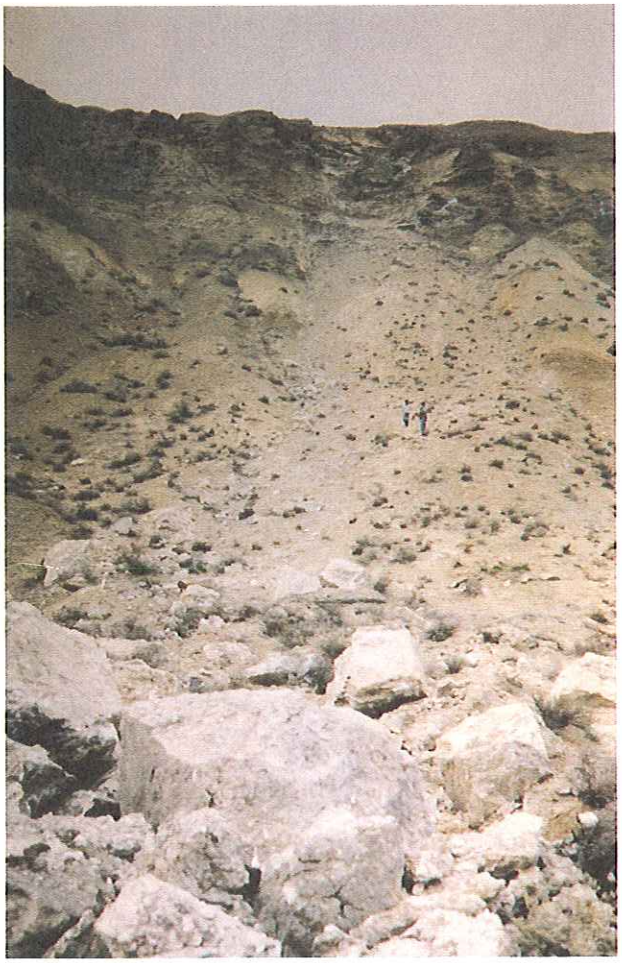
Recommend the stabilization program:____

List traffic control requirements during stabilization:____

List specifications to include in the contract:____

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 10



Describe the unstable conditions:

List the type(s) of failure:

List the cause(s) of failure:

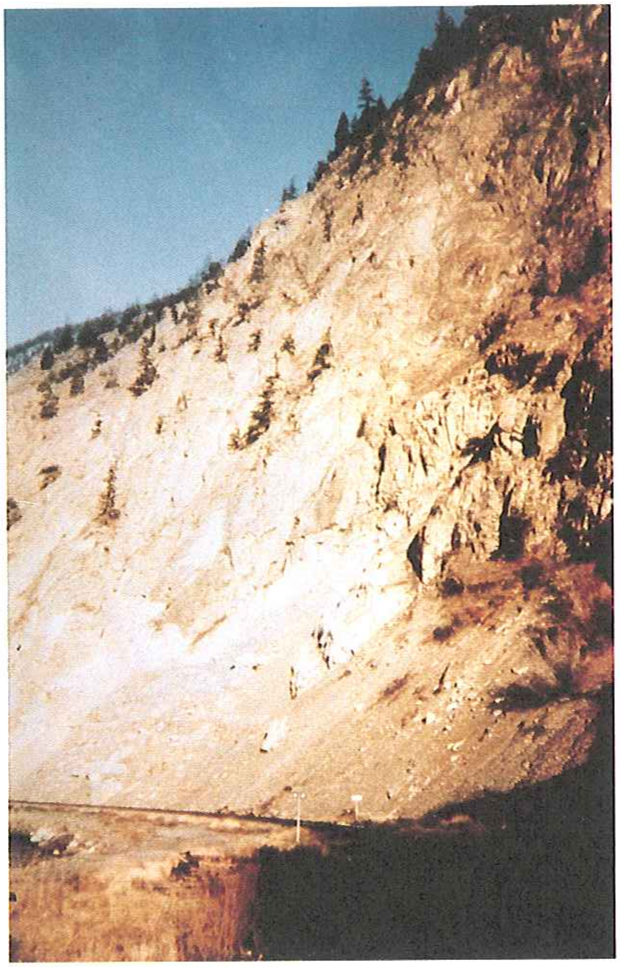
Recommend the stabilization program:

List traffic control requirements during stabilization:

List specifications to include in the contract:

MITIGATION EVALUATION EXERCISES

EXAMPLE NO. 11



Describe the unstable conditions:

List the type(s) of failure:

List the cause(s) of failure:

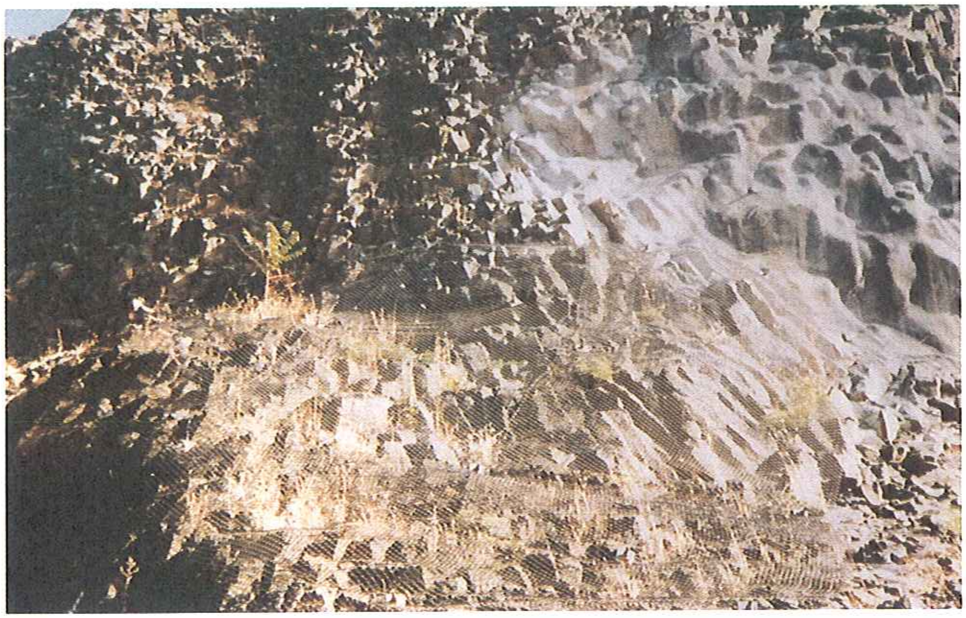
Recommend the stabilization program:

List traffic control requirements during stabilization:

List specifications to include in the contract:

<u>MITIGATION EVALUATION EXERCISES</u>

<u>EXAMPLE NO. 12</u>



Describe the unstable conditions:____

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List the type(s) of failure:____

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List the cause(s) of failure:____

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List traffic control requirements during stabilization:____

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List specifications to include in the contract:____

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Do you agree with the stabilization used? Why?____

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____

GLOSSARY

Acceleration A measure of force ($F = ma$). It is the time rate of change of velocity. It is measured in g's, the acceleration of gravity.

Acid Reaction Oxidation of sulphides in the rock which produces sulphuric acid and sulphate salts.

Active Bolt A bolt that is tensioned to increase the normal load on the potential failure surface.

Active Resistance Systems which increase the strength of the rock mass.

ADT Average daily traffic.

Aesthetics The visual appearance of a structure or site, particularly related to attractiveness.

Air Blast A sound pressure wave from a blast travelling through the atmosphere.

Air Photo Interpretation Interpretation of any photograph taken from the air, such as a photograph of a part of the earth's surface taken by a camera mounted in an aircraft, specifically of rock type, structural geology, topography and surface hydrology..

Alteration Any change in the mineralogic composition of a rock brought about by physical or chemical means, especially by the action of hydrothermal solutions.

Anchor Cable A high strength stranded cable installed in a borehole, anchored at one end and tensioned to increase the normal load on a potential failure surface. The tensioned cable is usually grouted.

ANFO Ammonium Nitrate-Fuel Oil Mixture. Used as a blasting agent.

Angle of Friction The angle above the horizontal of a plot of normal vs shear stress at failure.

Angle of Incidence The average angle that asperities make with the planar surface along a discontinuity.

Angular Velocity Velocity of the rock mass spinning around the center of the rock body.

Aperture Any of the various modifications in the exine of spores and pollen that can be used as a locus for exit of the contents.

Aquitard A confining bed that retards but does not prevent the flow of groundwater.

Asperities The surface roughnesses that occur along a discontinuity in rock.

Base Exchange The displacement of a cation of the insitu surface of a solid, as in silica - alumina clay mineral pockets, by a cation in solution.

Basket A platform held from a boom vehicle or crane from which stabilization work is performed.

Bearing Plate A special steel plate placed under the nut of a rock bolt to spread the load at the face of the rock.

Bedding Planes A rock formation formed by layering of rock as it was deposited, as in igneous flows or in separated sedimentary deposits. The visible separation of each successive layer.

Bench The horizontal ledge in a rock face. Benching is the process of excavating whereby terraces or ledges are worked in a stepped shape.

Bin Wall A support structure comprising metal, concrete or timber bins which are filled with free draining backfill.

Blast The operation of breaking rock by means of explosives. Shot is also used to mean blast.

Blasthole (Borehole) A hole drilled in rock or other material for the placement of explosives.

Blast Monitoring Measurement of the peak particle velocity and components created by blast detonation. Also measurement of air blast.

Block A large rock mass with a block shape. Also a type of rock failure where a block slides near horizontally.

Block Toppling Rock toppling failure where each topple is made up of successive blocks above each other.

Bonded Length The length of anchor unit that is grouted.

Boulders A detached rock mass generally larger than 10 inches.

Bounding Bouncing.

Brunton Compass An instrument used to determine directions, consisting of a magnetized needle suspended by the middle so that it is free to point to the magnetic north pole. Vertical angles can also be measured.

Buckling A type of failure in thinly bedded rock which is moderately to steeply dipping.

Buffer Holes Holes to be blasted between line holes and production holes using small weight of explosives to limit wall rock damage.

Burden The distance from an explosive charge to the nearest free or open face at the time the hole detonates.

Buttress A structure built under a rock overhang to strengthen or support the overhang.

Cable Lashing An isolated unstable rock stabilized by tying and tensioning anchored cables around the rock.

Catch Ditch A ditch at the base of a rock cut with a designed depth, width and shape to catch rockfall.

Cell Mapping Geologic mapping of structural discontinuities within a measured dimensional area (ie 20 feet by 20 feet).

Chain Link Mesh An interwoven wire mesh with small openings which is draped over a rock slope to control rockfall.

Chamfer Cut off at an edge or corner to make a slanting surface.

Charge Weight The amount of explosive charge in pounds.

Chemical Expanders A non-explosive demolition agent that breaks rock without causing noise, vibration, flyrock or environmental pollution. The expander consists of a special inorganic lime which when mixed with water exerts great expansion.

Circular A type of failure in weak rock which occurs along a curvilinear surface.

Claimant A person who makes a claim as a result of a rockfall event (legal).

Cleavage The property of a rock to split along aligned structures.

Clar Compass A special compass used to measure dip and strike of rock discontinuities.

Cohesion Shear strength of a rock not related to interparticle friction.

Colluvium An incoherent mass of soil or rock fragments deposited by rain wash or slow continuous creep usually collecting on or at the base of gentle slopes.

Compression A stress that causes a material (rock) to squeeze, compress and reduce in volume. If the stress is large enough the rock will fail.

Continuity The unbroken distance along a structural discontinuity.

Contorted Twisted rock structure.

Controlled Blasting Special blasting procedures such as presplit or cushion blasting employed along the final wall to reduce damage to rock slopes.

Core Recovery The amount of core recovered in a drill run, usually expressed as a percentage.

Crib A square or rectangular structure of timber, concrete or metal which is backfilled to provide support of soil or rock slopes.

Cushion Blasting The technique of firing of a single row of holes along a neat excavation line to shear the web between the closely drilled holes. Fired after production shooting has been accomplished.

Danger Trees Trees which exist on a rock slope or near the rock crest whose roots grow into discontinuities which under wind conditions can cause rockfall by levering the rock.

Deflection Berm A mound of soil or rock placed in a runout area to deflect or redirect rockfall to a safe location.

Deformation A change in the shape or dimension of a body resulting from stress and strain.

Delay The term used to describe a blasting cap which does not fire instantaneously but has a predetermined built-in lag or delay, time period.

Delay Blasting Blasting that uses delays or delay caps.

Design Standard The quality and detail design which is considered the standard of the industry at the date in question.

Designated Access Route The access route shown in the plans within the useable area of the environmental limits. Particularly for new construction or reconstruction.

Detonation An explosive reaction that consists of a shock wave through the explosive accompanied by a chemical reaction that furnishes energy to sustain the rock wave propagation with gaseous formation and pressure expansion.

Detour A bypass road constructed to carry traffic around a construction zone.

Differential Movement A variation in the magnitude of movement over short distances caused by change in stress in different materials, thickness or loading conditions.

Dip Angle The vertical angle between the plane of true horizon and the line of sight to the apparent horizon.

Discontinuity A structural geologic break in a rock, sedimentation sequence or separation of two unrelated rocks. (ie, joint, fracture, foliation, bedding plane, shear or fault).

Dissolution The process of changing into a liquid state.

Dowels A steel rod or bolt installed below or through rock to restrict movement of rock. The units are grouted but not tensioned.

Drag Scaling Removal of loose rock by dragging Caterpillar tractor tracks, timbers or other material up and down the slope using a crane.

Draped Wire Mesh Wire mesh draped over a slope to direct ravelling rock down into the highway ditch where it can be excavated.

Drill Cuttings Rock or soil particles that return in the drill fluid (air, water, mud) during drilling.

Dry Mix Process The process of providing shotcrete at the site where the final mixing water volume is added by the nozzleman.

Durability Ability to resist deterioration.

Dynamic Forces Forces which exert a sudden change in stress on rock. The stress application may be repetitive.

Electronic Distance Measurement (E.D.M.) A survey system where the distance is measured between an instrument and a mirror measuring the time span of a variety of waves which are reflected from the mirror.

Emergency Force Account A special account from which funds can be paid for unexpected changes in the work due to unexpected occurrences or requirements.

Empirical Based on experiment and observation as compared to theory.

Energy Analysis Evaluation of the magnitude and influence of variable energy application.

Energy Dissipating Mounds Mounds of soil or broken rock placed in the path of rockfall to slow the progress of the rockfall. Mounds are generally placed in several rows and staggered.

Entrained Air Small air bubbles entrained in cement, grout or shotcrete with the use of admixtures to increase durability, particularly to freezing.

Environmental Limit The area within which the contractor shall confine all construction activities, staging and stockpiling of materials and equipment.

Erosion Gradual wearing away of soil or rock by water, ice or wind.

Expansion Shell Metal wedges mounted on threads of the end of a rock bolt which are spread apart by turning the bolt to apply a very high lateral pressure on the rock so the bolt can be tensioned.

Explosive Any chemical mixture that reacts at high speed to liberate gas and heat and thus cause tremendous pressures.

Exposed Aggregate Natural gravel or rock facing which is bonded into the face of concrete walls or blocks.

Extensometers An instrument which uses wire or thin rods to measure the distance between two points.

Eye Bolt A bolt which has a circular head to attach rope, cables or mesh and is grouted into stable rock.

Fault A fracture or a zone of fractures along which there has been displacement relative to one another, parallel to the fracture.

Flagperson An employee who directs traffic.

Flexible Barrier A protective unit to catch rockfall that will yield upon impact.

Flexural Toppling Toppling failure where the rock slabs tip and fail in bending.

Flyrock Rock that is propelled into the air by the force of the explosion. Usually comes from prebroken material on the surface or upper open face. Flyrock is an indicator of wasted energy.

Foliation Planar or layered texture or structure in any type of rock.

Fracture A general term for any break in a rock. Fractures may be induced by construction, particularly by blasting.

Fragmentation The extent to which rock is broken into small pieces by primary blasting.

Free Fall That portion of rock movement not impeded in any way.

Freeze-thaw cycles The alternative condition of water in rock discontinuities freezing and thawing.

Frost Jacking The incremental widening of a discontinuity due to expansion created during freezing.

Gabions Wire baskets which are filled with cobbles to act as retaining walls.

Geologic Mapping The process of measuring structural geologic orientation and features of rock exposures.

Geotextile Membrane A thin membrane of plastic, rubber, PVC or similar material which may be reinforced to provide a separate layer, impervious layer or filter layer.

Glacial Till A range of soil to boulder sized material deposited by glaciers under, to the side or at the front of the glacier. The till may range from loose to high density.

Glaciosostatic Adjustment in tectonic stress due to the loading or unloading of glacial ice.

Global Position System (G.P.S.) The determination of ground or air coordinates using satellite distance measurement.

Gouge Fault infill caused by the breakdown of the adjacent rock during tectonic movement along the fault.

Groundwater All subsurface water.

Grout A thin mortar or chemical that is injected into drill holes to harden and provide bond and corrosion protection for rock bolts and anchor cables.

Guardrail A metal, timber, concrete or cable structure installed along a highway shoulder to prevent vehicles from accidentally departing the highway.

Hazard Rating A location by location system to evaluate the potential seriousness of rockfall on vehicles and occupants to assist in sequencing a state wide, rockfall mitigation program.

Hollow Core A small continuous opening the length of a rock bolt through which air or grout is pumped.

Horizontal Drains Perforated plastic pipe installed in near horizontal drill holes to provide underground drainage. The drains may be connected to a vacuum to increase flow and stability.

Hot-cold cycles Temperature variation cycles which cause alternative expansion and contraction of the rock and associated cyclical stress variation.

Hydraulic Conductivity The rate of fluid flow through soil or rock, generally expressed in cm/sec.

Hydraulic Splitting The breakage of rock by applying very high water pressure in a drill hole.

Hydrodynamic Shock The passage of seismic shock induced by blasting in rock below the water table.

Hydrostatic Pressure Water pressure which exists in discontinuities, cracks or pore space in the rock.

Hydrogeology The science that deals with subsurface waters and with related geologic aspects of surface waters.

Ice Jacking Ongoing opening of cracks or discontinuities in rock by freeze cycles. Also ice wedging.

Impact Energy The Kinetic energy of rock against a barrier or resisting fence.

Impact Zone The point at which a rock strikes the ground or a barrier.

Infilling Materials Materials which are created, deposited or washed into rock discontinuities.

Infrared Photography Specialized photography using the electro magnetic spectrum ranging in wave length from 0.7 mm to about 1 mm. Shallow groundwater can be located on infrared photos.

Interbeds A typically thin bed of one kind of rock alternating with beds of another kind.

Intersection Dip Angle The dip angle down the intersection of two discontinuities.

Jersey Barrier A concrete guardrail generally used to retain vehicles on the highway. Also used to stop rockfall on the boulder.

Joint The surface of fracture or parting in a rock, without displacement.

Joint Sets A group of more or less parallel joints.

Joint Roughness Coefficient A measure of the roughness of the surface of a joint developed by Barton.

Jute Mesh A burlap mesh placed on soil and weathered rock slopes after seeding to retain the seed and roots during rainfall.

Key Block A single rock which supports numerous rock above. The removal of this rock will result in the upper rocks failing.

Kinetic Energy The energy which the body possesses because of it's motion; in classical mechanics equal to one-half of the body's mass times the square of it's speed.

Lattice Type Minerals Minerals with a preferred orientation of crystallographic axes or planes.

Launching Pads Benches or promontories from which rockfall is projected a greater than normal distance when rolling or bouncing.

Lifters Near horizontal drill holes in which explosive is placed and detonated to break rock at the toe of a slope.

Limit Switch A control switch which activates a signal light or warning device when movement reaches a preset limit.

Line Drilling Closely spaced blast holes located and detonated along a predefined excavation limit.

Line Mapping Structural geologic mapping along a uniform elevation on a rock face.

Lithology The description of rocks on the basis of such characteristics as color, mineralogic composition and grain size.

Litigation The act of carrying on a lawsuit.

Lock Block Wall A retaining wall constructed of large concrete blocks which interlock top and bottom.

Mapping Sheet A form on which structural geologic details are written or printed.

Mechanical Anchor An anchor which expands at the end of a rock bolt in a drill hole to allow development of tension in the bolt.

Mesh Woven wire material used in rock restraining fences, draped mesh or gabion baskets.

Mill Scale A crusty coating that develops on steel materials.

Mitigation To control or improve stability of potential rockfall.

Moment of Inertia The sum of the products formed by multiplying the mass, or sometimes the area of each element of a figure, by the square of it's distance from a specified line.

Mud Capping A charge of explosive fired in contact with the surface of a rock after being covered with a quantity of mud, wet earth or similar substance.

Negligence (legal) Failure to take due care, as required by law, resulting in damage to property or injury to persons.

Non-Shrink Grout Grout that has an additive added to ensure shrinkage of the grout does not occur.

Nozzleman The person who operates the shotcrete gun to apply shotcrete and control the volume of materials at the nozzle.

Orientation The act of establishing the correct relationship in space of structural discontinuities.

Ortho Photography Avioplan photographs which use digital object date to produce automated photogrammetric distance measurement and maps with great accuracy.

Overblast The condition resulting from using more than the necessary amount of explosives which causes excess fragmentation, flyrock and noise.

Overbreak Excessive breakage of rock beyond the desired excavation limit.

Oxidation The combining of oxygen with another element to form one or more new substances.

Particle Velocity The velocity at which the earth vibrates, measured in inches per second due to an explosive detonation.

Passive Bolt A rock bolt which is grouted but not tensioned.

Passive Resistance Systems which offer resistance to rock movement.

Patrol An inspection usually in a vehicle along a highway to observe unsafe conditions that require attention.

Peak Particle Velocity The maximum particle velocity, caused by a blast in rock.

Peak Strength The maximum shear strength along a specific plane or surface of soil or rock.

Photogrammetry The science of obtaining reliable measurements from photographic images.

Photo Mosaic An assembly of aerial or space photographs or images who edges have been feathered and match to form a continuous photographic representation.

Phreatic Surface The upper elevation of the water table.

Pioneer Cut The initial rock excavation at the top of a high rock cut, generally with very limited work space.

Planar A type of rock failure which occurs along a dipping uniform plane rock surface.

Pore Pressure The fluid pressure, (generally water) measured at depth in soil or rock. Measurement is obtained with a piezometer.

Portal Tunnel entrance.

Pre-blast Survey A visual inspection and report of the existence of any cracks or structural damage in a structure prior to a nearby blast.

Presplitting Stress relief involving a single row of holes, drilled along a neat excavation line, where detonation of explosives in the hole causes shearing of the web of rock between the holes. Presplit holes are fired in advance of the production holes.

Pressure Treated Timber or wood which is impregnated with selected material to reduce rotting and insect borrowing. It may also increase fire resistance.

Production Blast Blasting of the main blast in front of final wall and buffer blast.

Proof Test A load applied to the stabilization unit (bolt or cable) in excess of the design load to ensure minimum support capacity is achieved.

Protection The installation of a structure or materials to catch rockfall before it reaches the highway or directs the rockfall away from or over the highway.

Rappel The technique of descending a cliff using a double rope around the climber to control the rate of descent.

Ravelling Rock that falls from a face as a generally ongoing occurrence.

Rebar Cage Reinforcing bars formed in the shape of a cage to provide extra strength in a concrete block or buttress.

Rebound shotcrete which bounces off the rock face and falls to the ground. This must be wasted.

Rebound Moduli The rate of which rock rebound occurs when an excavation in rock is made.

Recharge The process and amount of the addition of water to the zone of saturation, ie, groundwater replenishment.

Recutting The excavation of an additional slice of rock from a rock face.

Regional Stresses Tectonic pressures that exist regionally in the rock. These stresses usually vary in all directions.

Regolith A general term for the layer or mantle of fragmental and unconsolidated rock material, whether residual or transported and of highly varied character, that nearly everywhere forms the surface of the land and overlies or covers the bedrock.

Reinforced Earth A proprietary system of supporting earth and broken rock with a vertical or near vertical face. The structural face is supported by multi-steel slats at various elevations buried in the backfill.

Relaxation Removal of soil or rock reduces the load below or adjacent and results in some rebound movement.

Residual Strength The minimum shear strength available along a designated failure surface which occurs after some amount of movement.

Right-of-way Limits of state owned property within which a traffic corridor is constructed.

Rock Armour Large rocks placed to protect a slope against erosion.

Rock Bolts Tensioned steel reinforcement installed in a drill hole in rock to increase the normal load on a discontinuity and increase strength and stability. The bolt will usually be grouted to provide corrosion resistance.

Rock Breaker A pile driving type hammer operated from a backhoe unit to continuously impact rock to break it.

Rock Bridges Intact rock between discontinuities.

Rock Cut An excavation in rock for construction of a highway.

Rock Mass Characteristics Geotechnical parameters of a volume of rock which includes intact rock as well as the influence of discontinuities.

Rock Mass Failure Failure in rock which occurs through intact and discontinuous rock.

Rock Quality Designation (R.Q.D.) A measure of joint spacing and frequency in rock developed by Deere.

Rock Shed A steel, timber or concrete structure which passes rockfall and rock slide material over top a highway.

Rock Staining The application of an environmentally safe material which will change the color or appearance of a rock face.

Rockfall The relative free falling movement of a newly detached segment of bedrock or boulder.

Rockfall Records The systematic recording of rockfall events including location, size, damage, climatic conditions or cause.

Rockfall Trajectory The travel path taken by a rockfall.

Rockfall Velocity The velocity of a falling rock.

Root Prying The leverage of trees and roots due to wind within rock discontinuities in which the roots grow.

Runout The distance from the toe of a slope that a rockfall travels.

Safety Factor The ratio of resisting forces to driving forces.

Scaled Distance Factor of distance and quantity of explosive which relates to seismic disturbance.

Scaling The removal of loose rock by hand, with hand tools or by dragging a weight along a slope.

Schistosity The foliation in schist or other coarse grained crystalline rock due to the parallel, planar arrangement of mineral grains.

Seam A stratum or bed of mineral. Also, a stratification plane in a sedimentary rock deposit.

Secondary Blasting Breakage of large rock by blasting which did not break during the initial blast.

Seepage The movement of water or other fluid through a porous media such as soil or rock.

Seepage Forces The frictional force developed by seepage flow through soil or rock.

Seismic Acceleration Force The shock force developed by blasting.

Shear Strength The internal resistance of a body due to shear stress including frictional resistance and cohesion.

Shotcrete Concrete which is sprayed onto a rock surface in a thin layer to increase surface cohesion and reduce weathering.

Sight Distance The maximum distance over which a driver can see a standard size vehicle coming from the opposite direction.

Silica Fume An admixture to shotcrete which increases shear strength, viscosity and durability.

Site Specific Conditions are specific to each location and any design and construction must be developed accordingly.

Slaking The crumbling and disintegration of rock upon exposure to air or moisture.

Soil Mechanics The science of the application of the principles of mechanics and hydraulics to engineering problems dealing with the behavior and nature of soils.

Sovereign Immunity The legal condition whereby a state cannot be sued.

Split Set Bolt A proprietary rock bolt which increases rock stability using frictional resistance along the bolt.

Stabilization The improvement of rock stability by reducing the driving forces or increasing the resisting forces.

Staging Area The location of a contractor assembly, storage and supply area.

Steel Fibers Small fibers of steel mixed randomly in cement to increase the tensile strength of shotcrete.

Steel Strapping Strips of steel extending between adjacent rock bolts to support material between the bolts.

Stemming The inert material such as drill cuttings placed in the collar of a blast hole used to confine the gas products formed on explosion.

Stereographic Projection The projection of the perpendicular to a plane (the pole) of a structural geologic discontinuity on a half sphere.

Strain Meter An instrument which measures the strain or movement between two points.

Stratification Planes within sedimentary rock deposits formed by interruptions in the deposition of sediments.

Stratigraphy The branch of geology that deals with the origin, composition and arrangement of strata in a region.

Stress Relief The release of strain due to removal of an external load.

Strike The direction or trend taken by the horizontal projection of a structural surface.

Structural Geology The branch of geology that deals with the form, arrangement, and internal structure and especially with the discontinuities in the rock mass.

Surface Hardness Characterization of the hardness of the rock and slope surface.

Surface Roughness Characterization of the raggedness of the slope.

Swellex Bolt A proprietary rock bolt installed to increase rock strength using frictional resistance along the bolt.

Swelling Pressures The pressure developed due to expansion of certain clays (smectites) on the absorption of water.

Talus Rock fragments of any size or shape derived from and being at the base of a cliff or steep slope.

Technical Rock Climber A person trained to climb, rappel and scale rock slopes.

Tension The state of stress which tends to pull a body apart.

Tension Crack A crack that has developed in rock due to tension stress.

Texture The general physical appearance of a rock including the geometric aspects of its component parts and crystals.

Thermal Gradient The rate of change of temperature with distance.

Thin Section A fragment of a rock ground to a thickness of 0.03 mm and mounted between glasses as a microscopic slide.

Threadbar A bolt that has been threaded.

Tieback Wall A retaining structure that is supported by tying the wall with bolts or cables to the rock behind.

Tiltmeter A mechanical instrument used to accurately measure the change in tilt orientation of a rock.

Toppling A type of failure whereby steep dipping slabs tip toward an excavation.

Torsion The act of twisting one end of an object while the other end is held fast or twisted in the opposite direction.

Tort (legal) Any civil wrong for which the law requires damages.

Transient Water Pressure A water pressure which changes with external events - ie heavy rainfall.

Trial Blast An initial blast detonated to observe the result from which changes may be made to affect improved results.

Trim Shot A specially drilled and detonated blast to remove a rock nose or sculpture a rock face.

Transmissivity The rate at which water of the prevailing kinematic viscosity is transmitted through a unit width of the aquifer under a unit hydraulic gradient.

Tree Levering The crowbar effect of wind on a tree transmitted to tree roots which the roots have grown into rock discontinuities.

Tunnel An artificial underground opening through a hill or mountain to carry highway traffic.

Turnbuckle A device used for connecting and tightening sections of cables.

Unconfined Compressive Strength A rock mechanics test to determine the strength of a rock sample in compression without lateral support.

Vacuum Drainage The application of a vacuum to horizontal drains or tunnels to increase the rate of drainage from a slope and increase stability.

Value Engineering A bidder may submit an alternative design and bid that design on a project which he believes will save money and meet project requirements.

Vegetation Removal Line The area within which the contractor shall be allowed to remove the existing vegetation to facilitate construction.

Viaduct A bridge with a series of short spans.

Vibration Rapid alternating movement in alternate directions.

Viscosity The property of a substance to offer internal resistance to flow; its internal friction.

Warning Fence An electrified wire fence which if broken by a rockfall will signal an alarm to warn oncoming traffic.

Warning Sign A sign which provides notice to vehicle operators of some abnormal condition ahead for which extra alertness is required.

Weathering The destructive process by which rock materials on exposure to atmospheric agents are changed in color, texture, composition, hardness or form with little or no transport of the altered material.

Wedge A type of failure involving rock bounded on two sides by discontinuities which form a wedge configuration.

Wet Mix Process The process of providing shotcrete at the site which has all materials mixed in a central or ready-mix plant prior to application.

Wet-dry cycles The alternate cycling of wet and dry weather with associated change in stress conditions in rock slopes.

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